NONLINEAR PUSH-OVER ANALYSIS OF FLAT SLAB BUILDINGS WITH AND WITHOUT SEISMIC RETROFITTING

A Thesis Submitted
in Partial Fulfillment of the Requirements
for the degree of
Master of Technology

by

Tirandas Srikanth

to the
DEPARTMENT OF CIVIL ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY, KANPUR

July 1999

To

My Parents and Sisters

CERTIFICATE

It is certified that the work contained in this thesis entitled *"Nonlinear Push Over Analysis of Flat Slab Buildings With and Without Seismic Retrofitting"* by Mr. Tirandas Srikanth, has been carried out under my supervision and that this work has not been submitted elsewhere for a degree.

July, 1999

(Dr. Sudhir K. Jain)
Professor
Department of Civil Engineering
Indian Institute of Technology Kanpur

Abstract

Many existing flat slab buildings may not have been designed for seismic forces. It is important to study their response under seismic conditions and to evaluate seismic retrofit schemes. Two-dimensional nonlinear push-over analysis is carried out on a typical flat slab building. The building considered is designed only for gravity loads and wind loads. Comparison with similar conventional beam-column frames shows that the flat slab buildings have low lateral stiffness, low drift capacity and have hardly any ductility, while the overstrength is of similar order. The yielding of slab in sagging is significant in flat slab buildings because of insufficient bottom reinforcement. By making the column strip bottom reinforcement continuous, the yielding in sagging can be reduced considerably, but no improvement in drift capacity is obtained. Considering all these factors it is concluded that many existing buildings in high seismic regions may need retrofitting.

The performance due to retrofitting by a) column jacketing, b) addition of beams at floor, and c) column jacketing and addition of beams are also studied by push-over analysis. The retrofitting of ground storey by column jacketing is a good cost effective technique but is adequate only when seismic deficiency is small. The beam retrofitting reduces the sagging hinging significantly. Increasing the number of storeys of retrofitting by either column retrofitting alone or beam retrofitting alone does not improve the behavior significantly. With increase in number of storeys retrofitted with addition of beam alone, there is decrease in drift capacity, without much increase in strength. When column jacketing and addition of beam are adopted simultaneously on more number of storeys, large increase in lateral strength and stiffness can be achieved.

Acknowledgement

I sincerely express my deep sense of gratitude to my thesis supervisor, Dr. Sudhir K. Jain, for his valuable guidance, critical discussions and continuous encouragement. His aggressive approach of tackling problems has been a source of constant inspiration for me. I also express my gratitude to other faculty members at I.I.T. Kanpur, who taught me during the course work.

Sincere thanks are due to C. V. R. Murty for critical discussions and continuous encouragement. I also express my sincere thanks to Dr. Durgesh C. Rai for his help, and discussions during his visits to IIT Kanpur.

Sincere thanks are also due to all those school and college teachers, whose selfless encouragement brought me to this institute.

I am lucky to have a friend like Ramesh who always shared my grief. I am thankful to Ramesh and to Amar for their care and endless encouragement from distance.

The loving memory of my IIT Kanpur friends Sriram, Madhu, Ranjith, Naram, Piyush, Kiran, Jaswant, Dhiman, Sridhar, Roshan, Kiran YNVM, Turaga, Giridhar, Srinivas Kumar, Yugandhar, Kalyan, Srao, Rajesh, Mukul, Vishnu, Siddharth, Trivedhiji, Siju and Vijay will always be in my heart. The company of Bajpaiji, Pavan, Moitra, Manoj, Joshi, Jhaji and Navin will always be remembered.

I am thankfull to Motilal and Malkhan for their help during my thesis.

The help from Diptesh in final stages of thesis preparation is sincerely appreciated.

The financial support received from the Ministry of Human Resource Development (MHRD) is gratefully acknowledged.

Table of Contents

List of Tables

List of Figures

Symbols and Notations

α	Effective slab width factor
ϕ	Curvature
θ	Rotation
Δ_y	Yield displacement
Δ_{max}	Maximum displacement
θ_y	Yield rotation
θ_u	Ultimate rotation
ϕ_y	Yield curvature
ϕ_u	Ultimate curvature
f_c'	Specified compressive strength of concrete (cylinder)
C_1	Dimension of column in the direction of the moment considered
C_2	Dimension of column transverse to the direction of the moment considered
C_y	Base shear coefficient at first yield
C_w	Unfactored design base shear coefficient
C_x	Maximum base shear coefficient
d	Effective depth
DL	Dead load
E	Modulus of elasticity
f_c'	Cylinder strength
f_{ck}	Characteristic strength of concrete (cube)
f_y	Characteristic strength of steel
I	Moment of inertia
j_1, j_2, j_3	Empirical constants used in Baker's equation
k_1	Probability factor or risk coefficient
k_2	Terrian, height and structure size factor
k_3	Topography factor
l_1	Length of panel in the direction of moments
l_2	Length of panel transverse to the direction of moments
l_c	Distance of critical section to the point of contraflexure
LL	Live load
L_p	Length of plastic hinge
M	Moment
M_u	Ultimate moment
M_y	Yield moment
$M_{y\,hog}$	Yield moment in hogging
$M_{y\,sag}$	Yield moment in sagging
P	Axial load
P_{yc}	Ultimate uniaxial load capacity in compression
P_{yt}	Ultimate uniaxial load capacity in tension
p_z	Design wind pressure
V_b	Basic wind speed
V_z	Design wind speed at any height z
WL	Wind load

Chapter 1

Introduction

1.1 General

Flat slab is an American development, originated by C.A.P. Turner in 1906. It is a concrete slab reinforced in two or more directions so as to bring its load directly to supporting columns, generally without the help of any beams or girders. Sometimes beams are used where the slab is interrupted, as around staircases and discontinuous edges of the slab.

Flat slabs are often thickened close to the supporting columns to provide adequate strength in shear and to reduce negative moment. The thickened portion, i.e., the projection below the slab is called *drop* or *drop panel*. In some cases the section of column at the top, as it meets the floor slab or a drop panel, is enlarged so as to increase, primarily the perimeter of cross-section for shear. It is called *column head*. If no drop panel or column head is present, the system is sometimes referred to as *flat plate* rather than flat slab.

Failure of RC flat slab framing systems during severe earthquakes have led to widespread rejection of the flat slab as a viable system in regions of high seismicity. Economics and good performance under gravity loads have led to equally widespread acceptance of the system in the regions of lower seismic risk.

1.2 Behavior of Flat Slabs Under Lateral Loading

Flat slab buildings have considerably less lateral stiffness as compared to traditional slab-beam-column frames. Under gravity load also the slab deformation is

usually quite high because of the absence of beams. Low lateral stiffness of flat slab structures can lead to large earthquake induced motions in buildings, which can cause large P-Δ effects and result in damage to critical equipment and architectural elements.

Due to problems associated with excessive drift, inadequate punching shear and moment transfer capacities, flat slab structures provide poor seismic resistance, when compared with the conventional beam column framing. Hence, it is generally considered inadequate as a primary lateral load resisting system for multi-storied structures in regions of high seismic risk. Even when combined with a separate lateral load resisting mechanism, there is concern as to whether the connections possess sufficient lateral displacement capacity to survive expected lateral deformations.

Older flat slab buildings have performed poorly in moderate to high intensity earthquakes. Such structures were designed using building codes emphasizing gravity loading, without providing the strength, stiffness and deformation ductility needed to withstand cyclic motions. The connections were detailed to resist hogging moment, typical of gravity load moments at the connection, but not sagging moments arising due to lateral loads. Since the non-ductile slab-column connections in older buildings did not have slab bottom bars continuous through the interior column, the moment transfer capacity under positive bending is limited to flexural cracking strength of the slab. Under such cases brittle failures could occur at the connection during an earthquake. Hence it is generally accepted that these structures require retrofitting [Durrani et al., 1995 and Hayes et al., 1996].

Flat slab construction has a history of progressive failures. One of the most shocking was a 16-storey building nearing completion in Boston. Mitchell and Cook [1984] suggested remedies to arrest such progressive collapses. Fig. 1.1 illustrates the important role that the continuous bottom reinforcement can play after a punching shear

failure has taken place. After shear failure has occurred the top reinforcement rips out of the top surface of the slab and hence is ineffective. Therefore, a slab-column connection without bottom steel adequately anchored into the column region would have negligible post-punching shear resistance. In contrast, bottom reinforcement, when it is properly anchored into the column region, does not rip-out of the slab and therefore can provide significant dowel action, which can arrest the failure. If the bottom bars are well anchored and effectively continuous then they could provide an alternate load path in the form of tensile membrane once the slab-column connection is severely damaged. If an initial localized failure develops then the slab hangs off the columns by the effectively continuous bottom bars and distributed slab reinforcement. The surrounding slab regions provide in-plane horizontal restraint to the slab. Punching shear in the interior region of the structure would lead to two-way tensile membrane action while an edge panel would resist the loads by one-way tensile membrane action.

To ensure sufficient capacity to transfer the unbalanced moments, slab reinforcement needs to be anchored well into the slab-column junction. If the slab punching shear capacity is low, shear reinforcement has to be provided. But it is not always easy to provide shear reinforcement in slabs, because they are often very thin.

The slab-column connection involves a very complex behavior when subjected to lateral loads. Large shear forces and unbalanced moments are developed at the connection. The unbalanced moment is transferred to the column by a combination of flexure, shear and torsion. This is associated with cracking of slab in flexure and torsion, and the tension steel undergoing yielding. These factors make the flat slab still more flexible.

It is generally believed that the punching shear strength of interior slab-column connections is more than that of exterior columns. Because of confinement through the

surrounding slab it is also believed that the punching shear strength is greater in lateral loading than under pure gravity loading.

The existence of openings in the vicinity of a column reduces the area of concrete that can resist transverse shear, which makes the slab column connection weaker and failure can be brittle. Such openings, unless they are carefully designed, can be hazardous under strong earthquake shaking. The unbalanced moment becomes very high under such conditions. Hence openings should be carefully located and measures should be taken to transfer unbalanced moment effectively to the columns.

1.3 Literature Review

1.3.1 Experimental Investigations

Tests were conducted on fifteen interior slab panels with gravity loading alone, varying the size and shape of column and amount of reinforcement [Vanderbilt, 1972]. It was observed that the punching shear strength is a function of column shape, size and slab reinforcement. Circular columns were found to have higher punching shear strength than square columns of equal periphery. This difference was attributed to the stress concentration at the corners of square columns.

Dynamic experiments of precast concrete flat slab building models were performed on shaking table [Shen et al., 1984]. The models were made with and without shear walls and were of one-tenth scale. It was concluded that the weaker parts of prefabricated flat-plate shear wall structure were located at the joints between shear walls and slab-column connecting joints. Tests showed that due to the inherent low stiffness of prefabricated slab column system, shear walls are required to provide adequate drift control and to resist seismic loading. The development of inelastic deformation was

bay slab-column subassemblies were subjected to the same cyclic lateral displacement routine while each supported a different superimposed slab load. Tests showed that increasing the slab gravity load significantly reduces the capacity of the connection to transfer unbalanced moment. The lateral drift that the specimen could attain prior to failure also reduced with increasing gravity load. It was observed that slab cracking around the connection as a result of the increased gravity load moments, reduces stiffness of the interior connection.

Three half-scale interior and exterior fiber-reinforced slab-column connections were tested under simulated earthquake-type loading [Durrani and Diaz, 1992]. The amount of fiber-reinforcement was varied from 30 to 90 kg/m^3 of concrete. The addition of fiber reinforcement greatly improved the ductility and energy dissipation capacity of all specimens. It also enhanced the shear capacity of the interior connections but did not affect the strength of exterior connections. A smaller fiber content had no effect on the failure mode of the interior connections but with increased fiber content the mode of failure changed from punching shear failure to that of flexural failure.

Four interior, two edge and two corner post tensioned slab-column connections were tested at 43 % of full scale under simulated biaxial earthquake loading [Cruzado et al., 1994]. It was found that the increased gravity load results in decrease in lateral stiffness, strength, and deformability of the structure. To improve the understanding of the retrofitted interior connections, two of the above specimens were repaired after testing and one was retrofitted before testing. These three specimens were retested and their behavior was compared with that observed for the original specimens. The connection repaired with high strength epoxy grout had stiffness and strength exceeding that of the original connection. The connection repaired with a column capital had stiffness, strength and deformability dramatically increased in comparison with the

original connection. The connection retrofitted before testing was provided with steel plate jacketing. It had stiffness, strength, and deformability similar to that obtained using the column capital, and superior to the original connection (Fig. 1.3).

One of the effective ways of strengthening older seismically weak flat slab structures may be to provide ductile steel bracing systems. Strengthening of RC slab-column frames with ductile steel bracing is investigated on a one-third scale two-bay two-story slab-column frame [Goel and Masri, 1996]. Tests showed hysteresis loops dissipating significant amount of energy. The punching shear strength and ductility of the slab-column connections also increased.

Three large-scale slab-column subassemblies under seismic conditions were tested, varying primarily the amount and type of punching shear reinforcement [Tegos and Tsonos, 1996]. The presence of this reinforcement was found to improve the seismic performance and reduce considerably the deterioration of specimens after reaching their maximum capacity. It also changed the type of failure from punching shear failure to almost flexural failure. Inclined steel bars and steel fibers were found to increase the ductility and the energy dissipation capacity of the subassemblies. Moreover, the increase in punching shear capacity with inclined 45^o bars was greater than that obtained by steel fibers. The study proposed that design codes must specify minimum shear reinforcement for slab-column connections in order to avoid the brittle punching shear failure.

The effect of viscoelastic dampers (VED) was observed by fitting them to a one-third scale model of a flat slab building which was tested on the shaking table [Hayes et al., 1996]. The VED's are mounted in diagonal braces that extend between columns in a structure (Fig. 1.4). Earthquake induced tensile and compressive forces that develop in the braces are carried in direct shear by VED. The addition of VED in braces both stiffens the structure and enhances the energy dissipation capacity. The increased

damping reduces structural response to ground motion. While the use of supplemental VED is conceptually effective, most prior research has focussed on their use in steel structures. There is concern that for VED to deform sufficiently to utilize their energy dissipation characteristics, the parent RC structure should have deformed enough. This results in significant cracking associated with strength and stiffness loss in the parent structure before VED's start dissipating energy.

Six specimens of high-strength concrete slab-column connections with shear reinforcement and four specimens with shear studs as shear reinforcement were tested [Salakawy et al., 1998]. Four of these ten specimens had openings. The benefit of placing high-strength concrete in slab is to increase the strength of slab-column connections. This is an attractive option in cases where the slab perforations have significantly reduced the available section for shear transfer. The parameters varied were the location and size of the openings, and the amount of shear reinforcement. The specimens with opening located along the line of the unbalanced moment were found to have less shear capacity than that with opening located transverse to the line of action of the unbalanced moment. Shear reinforcement was effective in increasing the shear capacity of the connections by 20 to 32%. The performance of connection was much better with shear studs than with shear reinforcement. Shear studs changed the mode of failure from shear failure to flexural failure. In the case of the connection with a large opening, the shear studs increased the strength, where as the shear reinforced connection failed in punching mode.

1.3.2 Analytical Studies

Over the years, efforts have been made to develop simplified analytical models for flat slabs. However, because of the three-dimensional action and the complexities involved in the actual behavior, these models have been slow to develop. The analytical

studies involve two basic issues 1) capacity of slab-column connections, and 2) frame analysis of flat slab systems.

1.3.2.1 Slab-Column Connection Capacity

a) Eccentric shear stress model

This is a method based on an investigation by Hanson and Hanson [1968]. Later it was reviewed in the ASCE-ACI Committee 426 report [1974]. It predicts the punching shear capacity for a given level of unbalanced moment for all types of connections. The critical section for shear is assumed to be at a distance d/2 from the face of the column. When drop panel and column head are provided, the section at a distance d/2 from their face is also taken as critical section. The shear stress at each of these sections has to be checked. The shear stress caused due to unbalanced moment is assumed to vary linearly with distance from centroidal axis. It varies about the centroid of the critical section, eccentrically about critical periphery (Fig. 1.5). Hence, this semi-empirical method adds to the gravity shear on one side and subtracts on other side.

Tests were conducted on slab-column connections subjected to gravity and lateral loading. The strength of these connections was compared with that calculated by eccentric shear stress model. It was concluded that the eccentric shear stress model does not correlate well with the measured strength of edge connections [Moehle, 1988]. When the ratio of gravity shear to pure shear strength of connection is less that 0.75, no significant interaction was found in tests. Hence, the model becomes excessively conservative for edge connections. Despite much criticism, many design codes continue to use this model because of its simplicity in application.

b) Truss model

The eccentric shear stress model assumes one particular critical section, which is vertically oriented at some distance from the column. Alexander and Simmonds [1987] strongly disagreed with any single critical section as the failure surface changes with the moment-to-shear ratio. They proposed a truss analogy model to predict the ultimate capacity of the connection. It consists of a three-dimensional space truss composed of concrete compression struts and steel tension ties (Fig. 1.6). The reinforcing steel and concrete compression fields are broken down into individual bar-strut units. This model represents the flow of forces and also predicts the failure mechanism. It considers the effect of diagonal cracking and explains the role that flexural reinforcement plays in determining the shear strength. Because of its complexity this model has not been adopted by the design codes.

c) Beam analogy model

The beam analogy model for predicting the strength of slab-column connections was introduced by Hawkins [1973]. This model has been developed based largely on extensive test results from interior, edge and corner connections. In this method, the slab adjacent to the column is considered to act as beams running in two directions at right angles framing into the column faces. The slab strips making up the beams are subjected to shear force, bending moment and torsional forces and redistribution of these actions is assumed to be able to occur between the beams. Each beam is assumed to be able to develop its ultimate bending moment, torsional moment and shear force, making due allowance for interaction effects. The strength of the connection is calculated by summing the contributions of the beams. Several possible combinations of shear, bending and torsion have to be considered, since not all the faces may reach their limiting strengths at the same time.

Hawkins [1973] assumed that stirrups only contribute to the torsional strength. However, Park and Islam [1976] proposed a simplified beam analogy model, which also evaluates the contribution of stirrups towards shear and torsional strengths.

d) Effective transfer width model

Moehle [1988] developed this model for estimating the strength of slab-column edge connections (Fig. 1.7). It is based on the concepts of diagonal compression theory [Mitchell and Michael, 1974]. He surveyed the experimental data on edge connections, and found that there is no significant interaction between shear and the moment applied. Either the connection fails in its pure shear strength, or at its pure flexural strength, whichever is reached first. Hence, in this model failure is assumed to occur when either of these is reached. Shear capacity is computed as a product of critical section and the critical shear stress capacity. Moment strength is computed as the flexural strength provided by reinforcement within an effective transfer width. He suggested an effective transfer width of C_2+2C_1 for moment transfer from slab to column, where C_1 is the dimension of column in the direction of unbalanced moment and C_2 is the dimension of column transverse to the direction of unbalanced moment. The method was found to correlate well with the experimental data, while the eccentric shear stress model was very conservative.

1.3.2.2 Frame Analysis

a) Effective slab width method

In this method, originally developed by Pecknold [1975], an effective width factor α is obtained such that a slab of effective width αl_2, subjected to uniform support rotation, would have a total support moment equal to that of the original slab of width l_2 and with varying rotation. Darvall and Allen [1984] developed effective width co-efficients for slabs with drop panels, noting that drop panel increases the stiffness which

in turn increases the effective width. Subsequent studies showed that effective width varies with slab aspect ratio, column sizes, and with change in reinforcement.

Once the effective width is fixed the complete three-dimensional model of the structure is analyzed as two-dimensional frame consisting of the original columns and a beam of original slab depth and effective width. In reality some areas may experience very little cracking, while other areas close to the connections may experience substantial cracking. It has been recognized by many researches that gross section properties overestimate the actual stiffness. The effect of cracking in the slab causes significant reduction in the section properties. Moehle [1984, 1992] suggested that using a stiffness reduction factor of 1/3 in conjunction with the effective beam width model leads to a reasonably good estimate of working load stiffness. Vanderbilt and Corley [1983] and Cano and Klingner [1988] also used 1/3 of the gross stiffness to account for cracking. Dovich and Wight [1994] have suggested effective widths for both stiffness and strength of slab-column connections, based on experimental research and analytical study (Fig. 1.8).

b) Equivalent frame method

A number of equivalent frame methods have been proposed to predict the moment distributions and stiffness expected in slab-column frames. Generally, these methods incorporate a transverse torsional member at the slab-column connection, which models the torsional stiffness of the slab adjacent to the connection. Corley et al. [1961] suggested the concept of transverse torsional members as part of a study, which later developed to a very well known, equivalent frame method of analysis. The torsional members are assumed to have only the torsional degree of freedom. The rotational stiffness of the equivalent column is found out as the function of the stiffness of original columns above and below the joint and the torsional stiffness of the transverse element.

This way the three-dimensional structure is represented by a series of two-dimensional frames. Corley and Jirsa [1971] modified the procedure for computing the stiffness of the torsional element as part of a study on pattern loading effects.

The most widely used equivalent frame method is the ACI code method. It is used both for lateral and gravity load analysis. However, it is only good for gravity and monotonic load. The researchers generally agree that it is not good for seismic loads because the stiffnesses used are not the intrinsic properties of the structure. Cano and Klingner [1988] and Vanderbilt and Corley [1983] have discussed the application, advantages and disadvantages of this method elaborately.

Elias [1979, 1983] has tried to combine the transverse element with the slab to give an equivalent slab and developed a stiffness matrix for the equivalent slab based on analytical studies.

1.4 Scope of the Study

The lateral behavior of a typical flat slab building which is designed according to IS : 456 – 1978 is evaluated by means of nonlinear push-over analysis. The inadequacies of these buildings are discussed by comparing the behavior with that of the conventional beam-column framing. The response is also studied when the flat slab system has continuous column strip bottom reinforcement as is now required by the ACI code. Further, the effect of retrofitting schemes by a) column jacketing, b) addition of extra beams, and c) column jacketing and addition of extra beams is also studied. The improvement obtained from each of these schemes is discussed.

1.5 Organization of the Thesis

The thesis has been organized in five chapters. This chapter reviews the literature on experimental investigations and analytical models developed for flat slabs. The behavior of flat slab buildings under lateral loading is also presented in this chapter. Chapter 2 describes the method adopted for push-over analysis. The description of the program used and procedure of analysis are presented. Chapter 3 presents the results of typical flat slab frame. Its behavior is compared with that of the beam-column frames. Results are also presented for a flat slab frame with continuous column strip bottom reinforcement. Chapter 4 presents the analysis for retrofitted flat-slab frames. A brief summary and important conclusions are presented in Chapter 5.

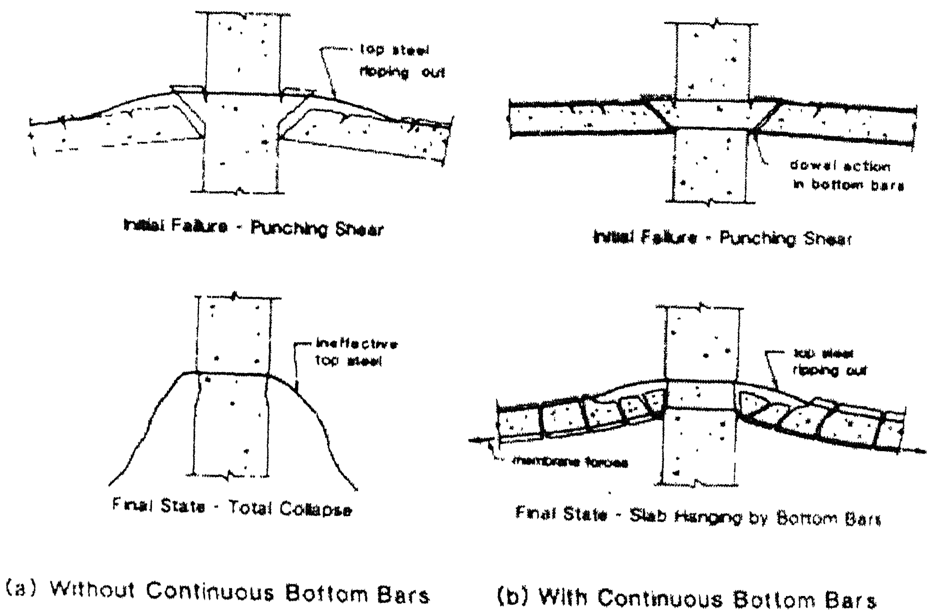
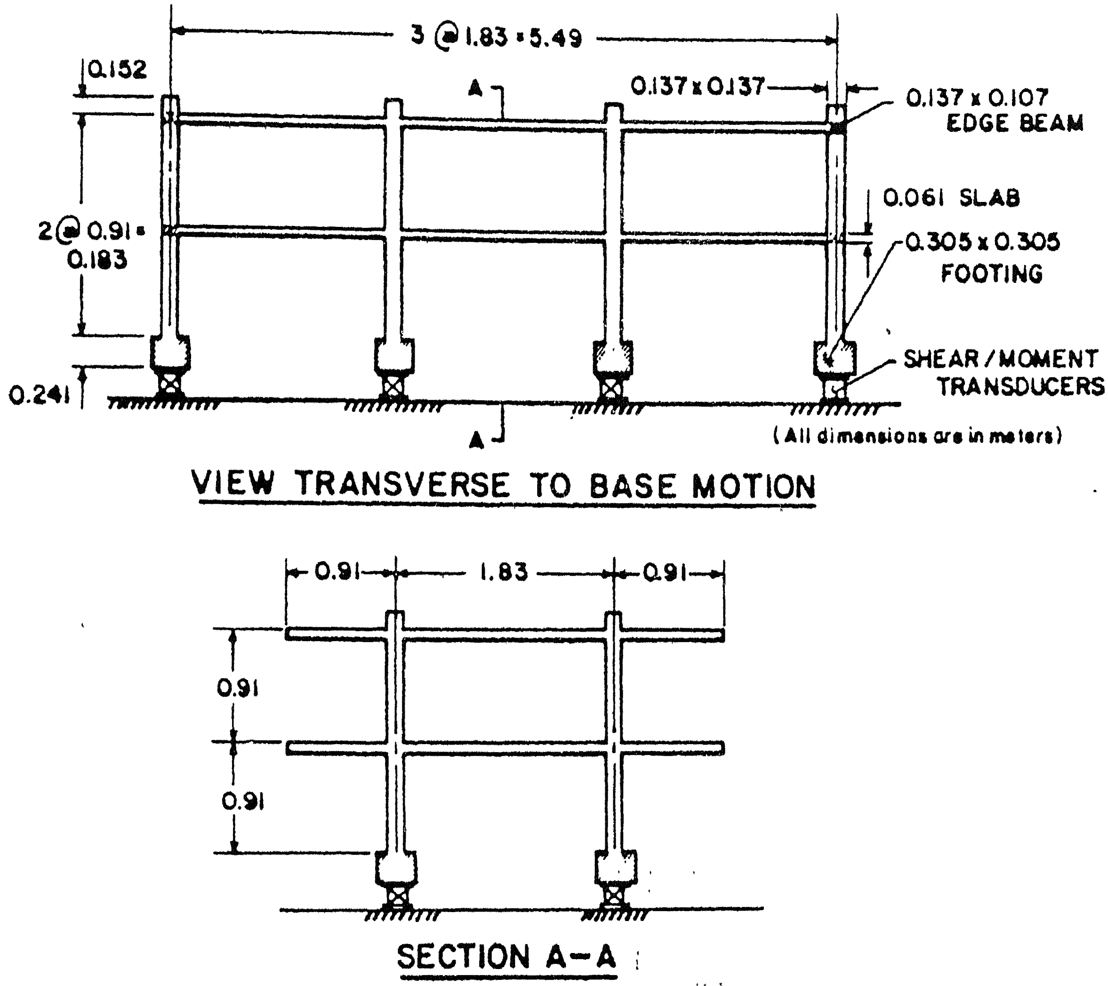
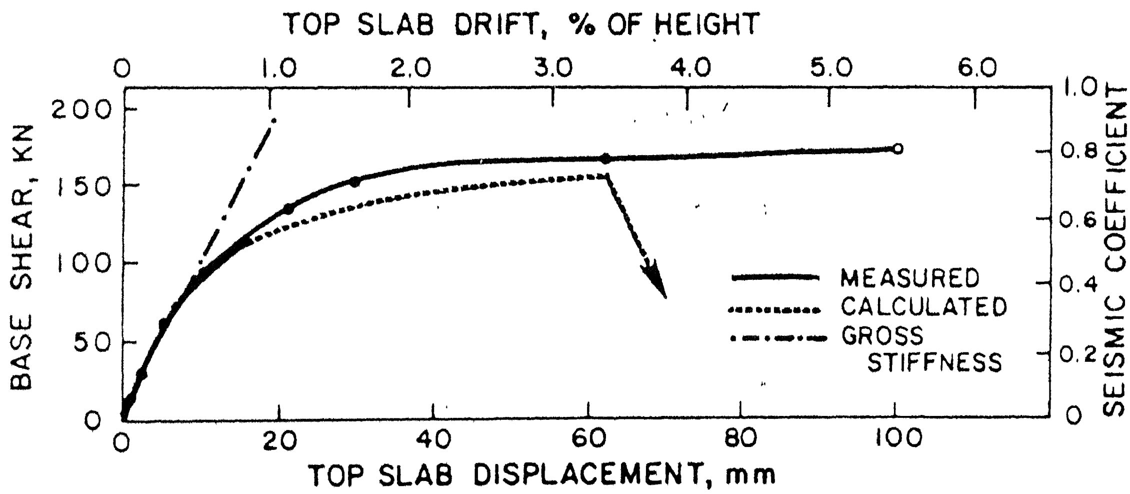


Fig. 1.1 : The Role of Bottom Reinforcement Anchored into Supports [Mitchell and Cook, 1984]

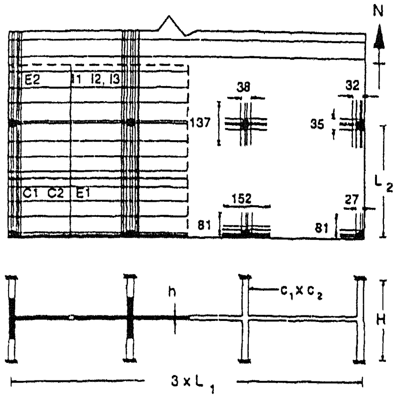


a) Configuration of Test Structure



b) Envelope of Top Floor Displacement and Base Shear

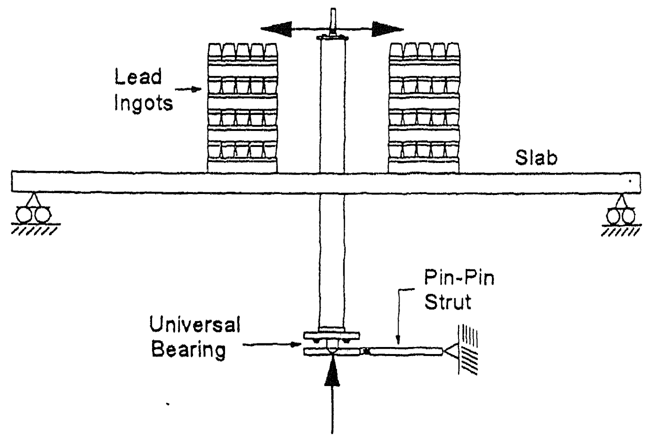
Fig. 1.2 : Experimental Study of Two-Storey Three-Bay Flat Plate Frame [Moehle and Diebold, 1984]



Connection	h	$c_1 = c_2$	$L_1 = L_2$	H
Interior	8.9	20.3	373	133
Exterior	9.2	19.5	366	131

All dimensions in centimeters

Model structure.



Test setup; east-west elevation

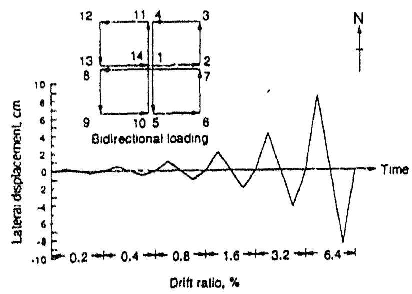
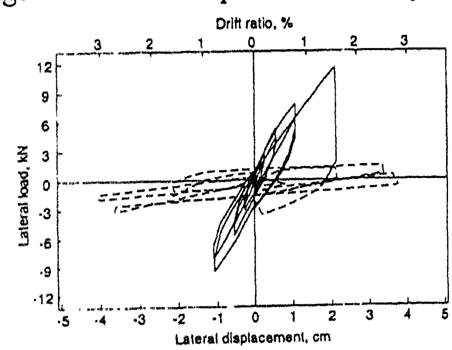
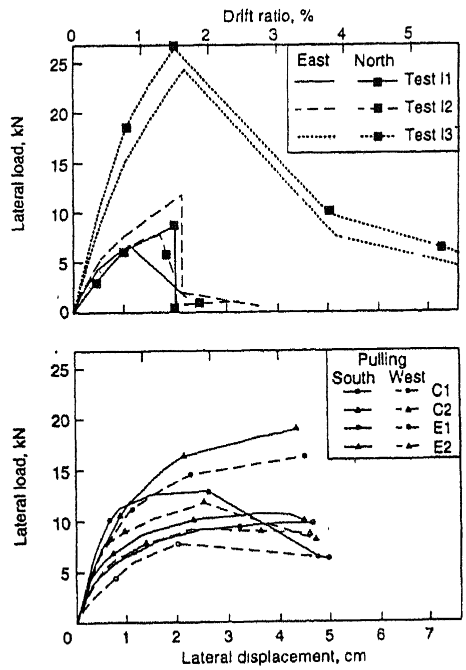


Figure 3. Lateral displacement history.



Load vs displacement; EW dir. Test I2.



Load vs displacement envelopes.

Fig. 1.3 : Post Tensioned Slab-Column Connections Under Earthquake Loading [Cruzado et al. 1994]

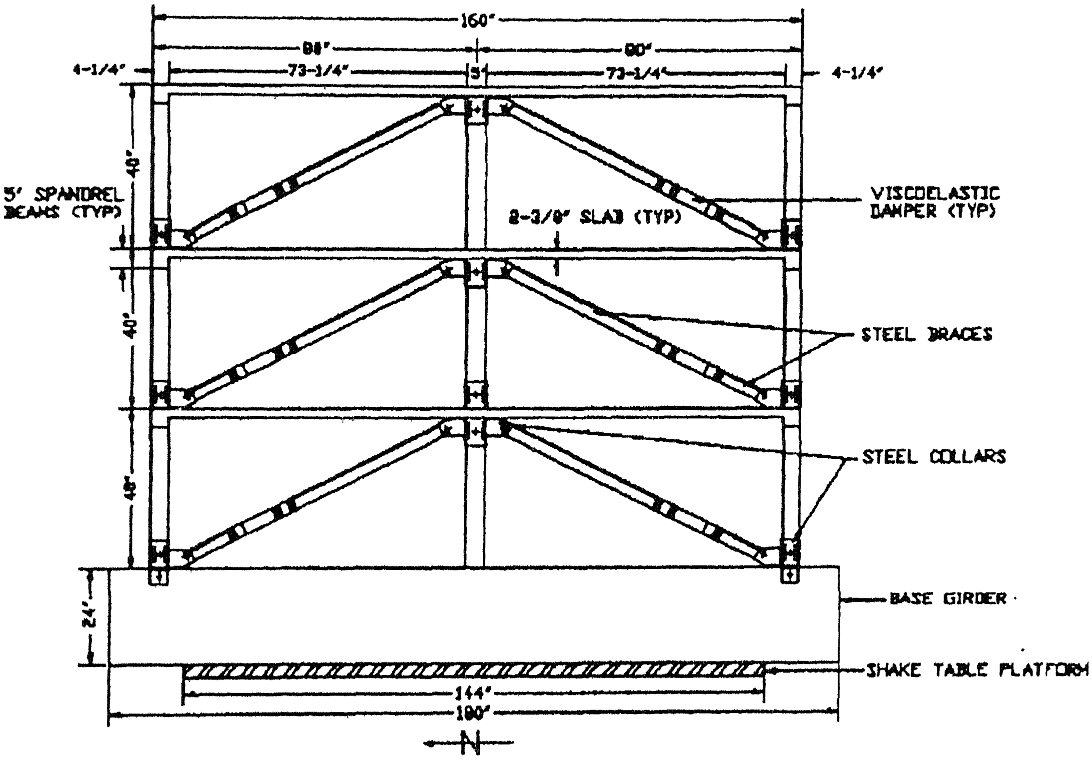


Fig. 1.4 : Model Structure Tested with Viscoelastic Dampers [Hayes et al. 1996]

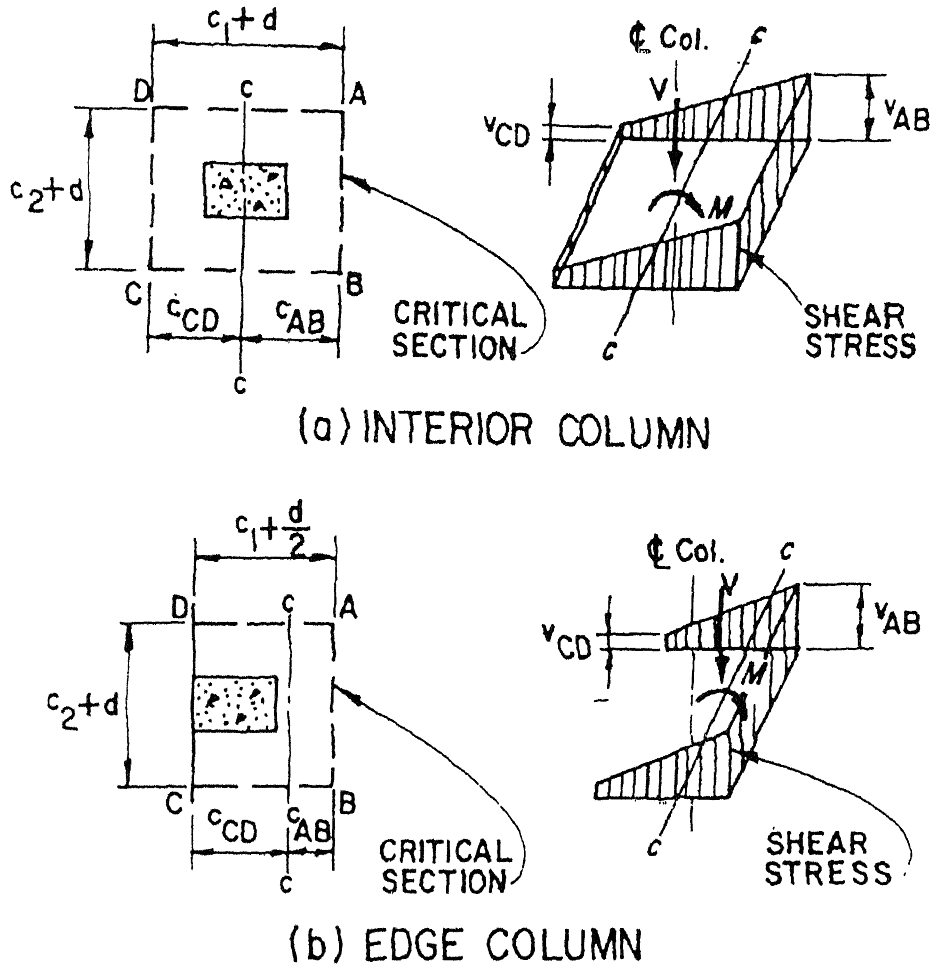


Fig. 1.5 : Eccentric Shear Stress Model (ACI 318-95)

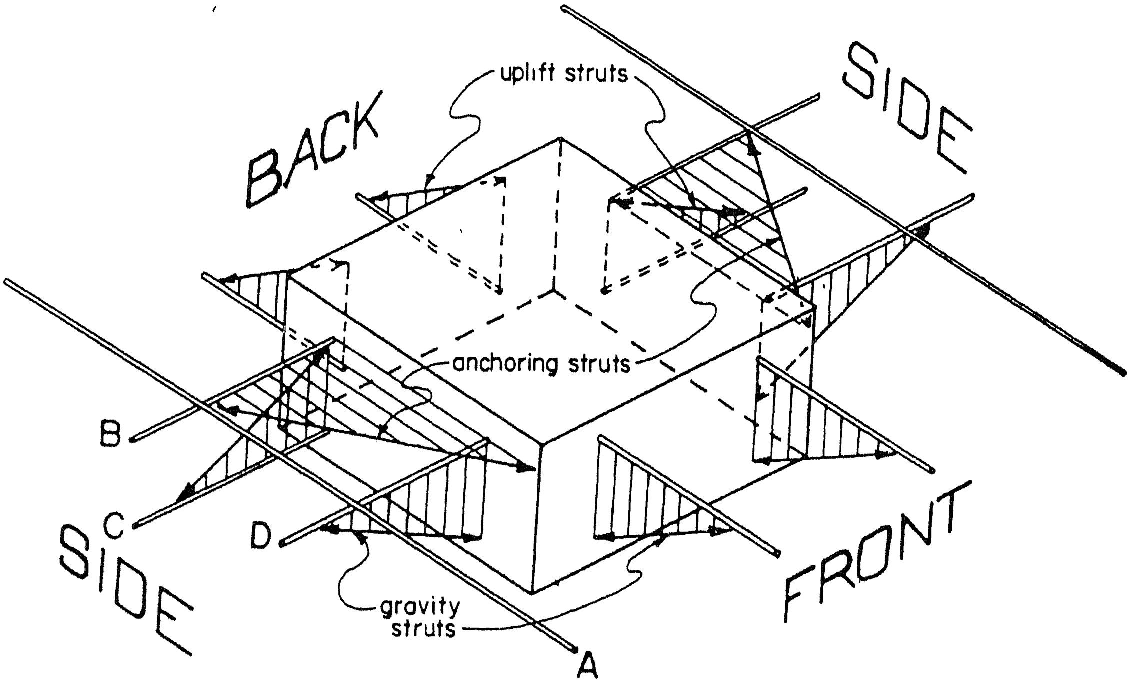
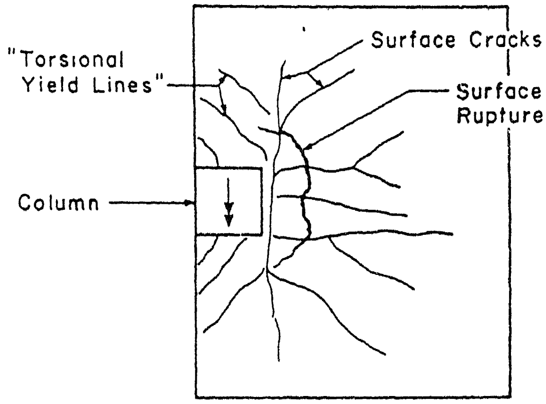
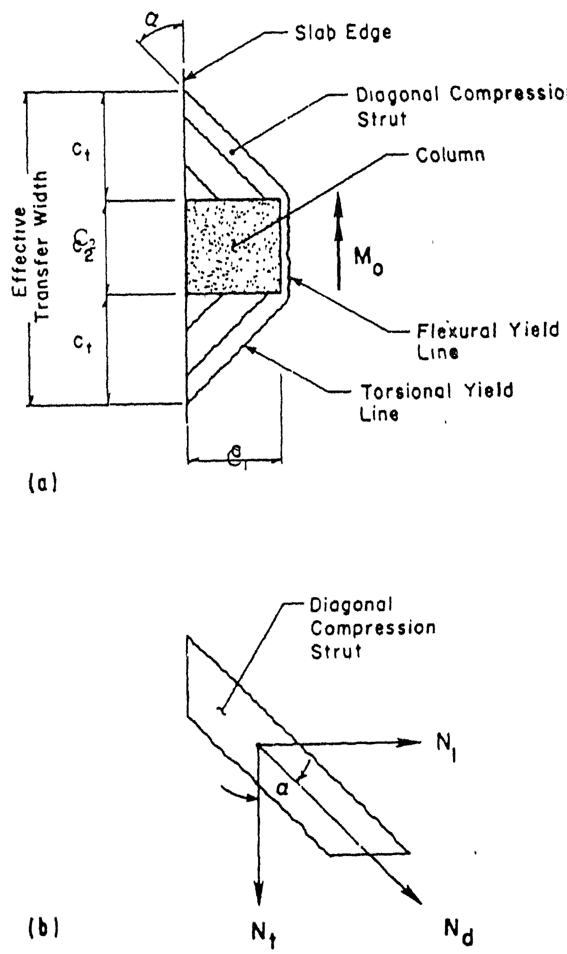


Fig. 1.6 : Truss Model [Alexander and Simmonds, 1987]

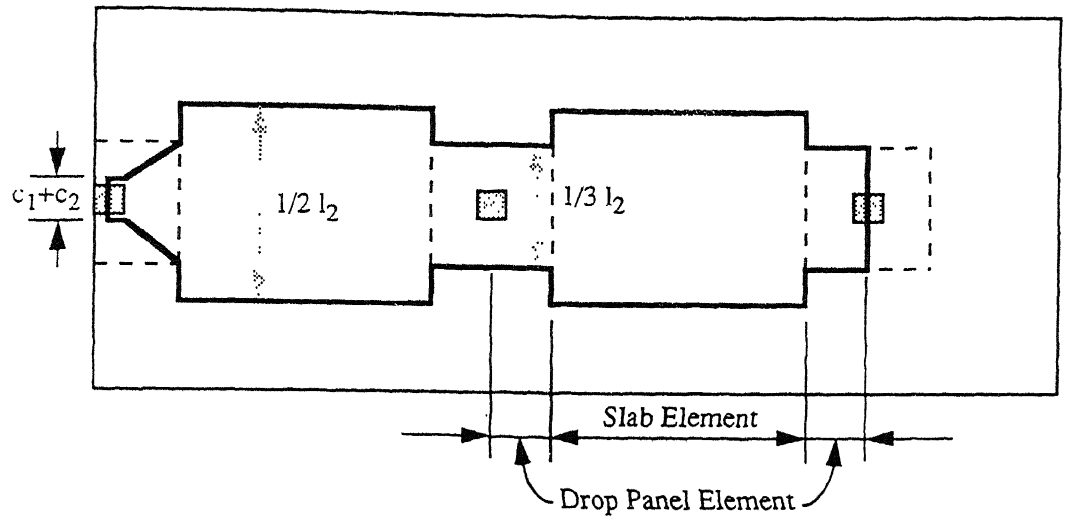


a) Typical Damge at Failure

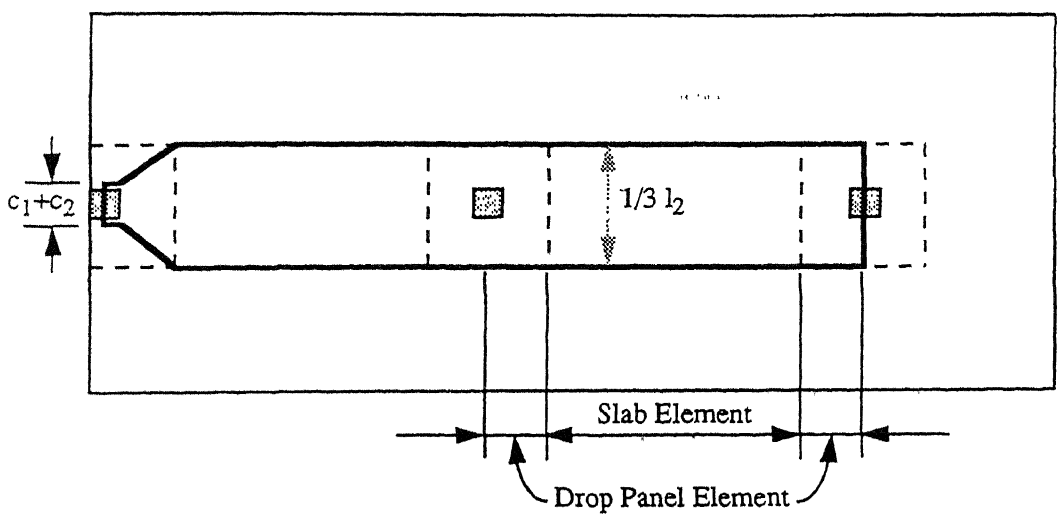


b) Illustration of Diagonal Compression Strut at Edge Connection

Fig. 1.7 : Effective Transfer Width Model for Edge Connections [Moehle, 1988]



(a) Effective Width For Strength



(b) Effective Width for Stiffness

Fig 1.8 : Effective Slab Widths Proposed by Dovich and Wight [1994]

Chapter 2

Push-Over Analysis

2.1 Introduction

Static push-over analysis is a procedure where a building model is subjected to increasing static lateral loads in one direction. These lateral forces continue to increase until the building collapses or a specified drift is reached. This procedure is very useful for studying the progression of damage in a building with increasing drift levels. It is not only an analytical test of the behavior of the lateral load resisting system, but also of other parts of the structure which may be affected by lateral drift or which may interfere with the lateral load resisting system. The changes in the slope of the base shear versus roof displacement relationship indicate yielding of various elements of the structure.

As yet there are no standard loading patterns established for the push-over analysis. In general, two types of lateral load distributions along the height of the frame are used. They are uniform and inverted triangular distributions (Fig. 2.1). One view is that since the mass on the floors is uniformly distributed, the lateral load distribution has to be uniform. Another view is that since contribution of the first mode is more, the lateral load distribution should follow the first mode. Most of the seismic design codes follow inverted triangular distribution for seismic design of buildings. This distribution creates more critical situation than the uniform distribution. For analysis in this study, the inverted triangular load distribution is used.

2.2 SNAP-2DX Modeling of the Structure

Kannan and Powell [1972] developed a general purpose program for dynamic analysis of inelastic plane structures at the University of California at Berkeley [DRAIN-

2D]. Later it was modified and expanded by Tang and Goel [1988] at the University of Michigan [DRAIN-2DM]. This was further modified by Rai, Goel, Firmansjah [SNAP-2DX]. The coding of element modules in this program is very similar to that of Kannan and Powell. But there are significant differences in solution strategies and other global modules.

The SNAP-2DX program is used in this study for the displacement-controlled non-linear push-over analysis of the building. The structure is discretised into two-dimensional beam and column elements connecting at the nodes. The analysis is done in several segments. In each segment it uses the step-by-step iteration to have the results converge within an allowable range.

The nodes at each floor level are constrained in the analysis so as to have equal displacements in the horizontal direction. The implication of this is to neglect the axial deformation in beam elements.

The element 2, "reinforced concrete beam column element" is used to model the beams and columns of the two-dimensional frames. The inelastic behavior is modeled by allowing the plastic hinges to form at the element ends. The behavior of the elements is specified by a bilinear moment rotation (M-θ) relationship. It has a certain initial stiffness till it reaches the yield point. Then the stiffness decreases and the M-θ relation continues to go on and on without any failure point. Hence, the analysis stops when a specified drift limit is reached or when a mechanism is formed in the structure. The yield points in sagging and hogging can be different. 'Element 2' considers same stiffness for both hogging and sagging moments, and same reduced stiffness after yielding (Fig. 2.2). The program approximates the "P-Δ" effect by adding a geometric stiffness to the elastic stiffness of the column. The geometric stiffness is based on the axial force in the element under gravity loading. The axial force-moment (P-M) interaction for the column element

is taken as shown in Fig. 2.3. The balance point on yield surface of P-M interaction diagram is assumed to be at (1.05 M_y, 0.12 P_y). This value is chosen because, in general, the individual balance points of all columns is found to be very close to this point.

The program does not allow the elements to be loaded along their length. Hence, the fixed end moments and reactions due to gravity load are applied to the elements at their ends.

2.3 Moment - Curvature and Moment - Rotation Relationships

The program developed by Mandal [1993] gives the M-ϕ curve of a given RC section with geometry, material, axial load and confinement. It uses the stress-strain relations recommended by IS 456: 1978.

The effect of axial load on M-ϕ relationship of the elements is significant and only a unique value of axial load can be given for evaluating the M-ϕ curve. This value is obtained by assuming that the axial load on columns is due to full design dead load and one-half of design love load.

The M-ϕ curve is idealized as bilinear curve as shown in the Fig. 2.4. A straight line is drawn, which starts from the origin and passes through the yield point of actual M-ϕ curve. A tangent is also drawn at the end of the actual M-ϕ curve. The point where these two lines intersect is taken as yield point in the idealized M-ϕ curve. The two straight lines represent the linear regions before and after yielding. The calculation of M-θ relationship from M-ϕ relationship is shown in Fig. 2.5. The moment diagram at a particular instant in an element due to lateral loads is as shown in Fig. 2.5 (a). This is replaced by an equivalent cantilever beam of half the length, with a point load at the end as shown in Fig. 2.5 (b). The rotation at yield point and ultimate point are calculated as

shown in Figures 2.5 (d) and (e). The length of plastic hinge L_p is calculated by Baker's formula [Park and Pauley, 1975] given by:

$$L_p = j_1 j_2 j_3 \left(\frac{l_c}{d}\right)^{1/4} d \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 4.1$$

where j_1 = 0.7 for mild steel or 0.9 for cold-worked steel, $j_2 = 1 + 0.5P_u/P_o$, P_u = axial compressive force in member, P_o = axial compressive strength of member without bending moment, j_3 = 0.6 when $f_c^/ = 35.2$ kN/mm^2 or 0.9 when $f_c^/ = 11.7$ kN/mm^2, $f_c^/$ is the cylinder strength. For other values of $f_c^/$ linear interpolation can be taken. Cylinder strength is taken as 0.8 times that of the cube strength. l_c = distance of critical section to the point of contraflexure, and d = effective depth of member.

2.4 Ductility and Overstrength

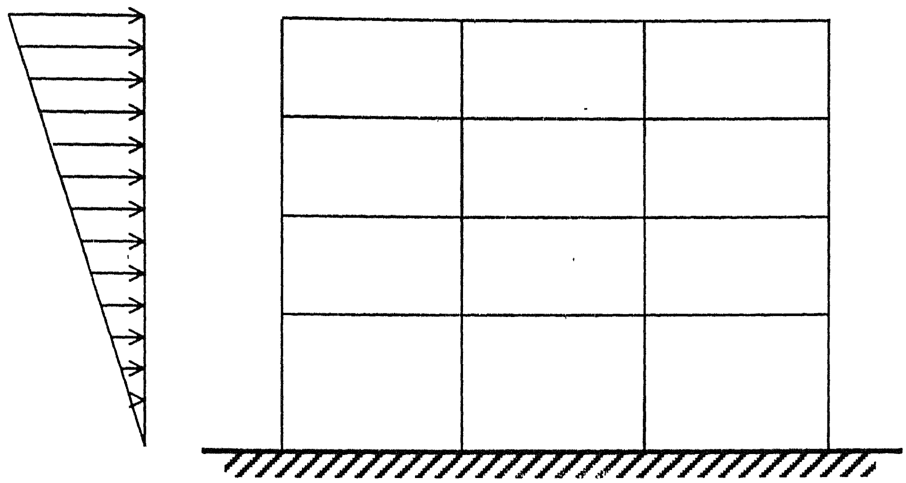
2.4.1 Ductility

The ductility of the structure is defined as the ratio of maximum overall displacement of the structure (Δ_{max}) to the displacement at the yield point of the structure (Δ_y). The Δ_{max} is taken as the least of a) displacement at which the first column reaches θ_u, and b) displacement at which specified drift limit (i.e., 2.5%) is reached. The yield point of the structure (Δ_y) is idealized as shown in Fig. 2.6. Two lines are drawn, one from the origin and another line from the end of the force-displacement curve. These two are drawn such that the dotted areas in Fig. 2.6 are equal. The intersection of these two lines is taken as the yield point.

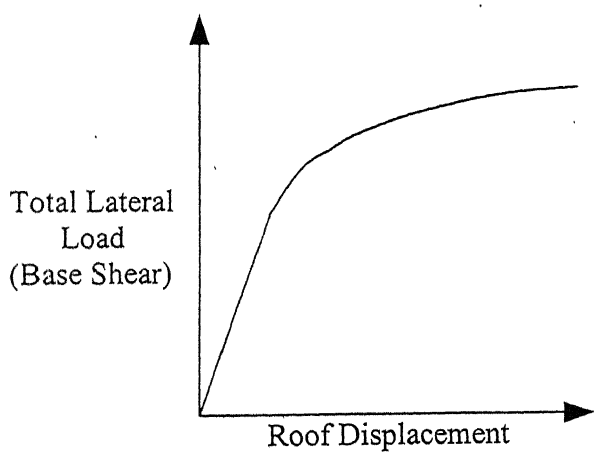
2.4.2 Overstrength

Overstrength of the structure is defined as the ratio of maximum base shear coefficient (C_y) obtained from response of structure to the unfactored design base shear coefficient (C_w) of the structure (Fig. 2.6). The reasons due to which this reserve strength

exists include: load factors are applied to the design loads, safety factors are applied to the material strength, actual material strengths are in general higher than the code specified values, the member sizes provided may be higher than the design values, reinforcement provided may be more than that required by the design and the provisions of minimum member sizes and minimum reinforcement requirements also increase the strength.



a) Lateral Load Distribution on the Frame



b) Force – Displacement Relationship

Fig. 2.1 : Push-Over Analysis Procedure

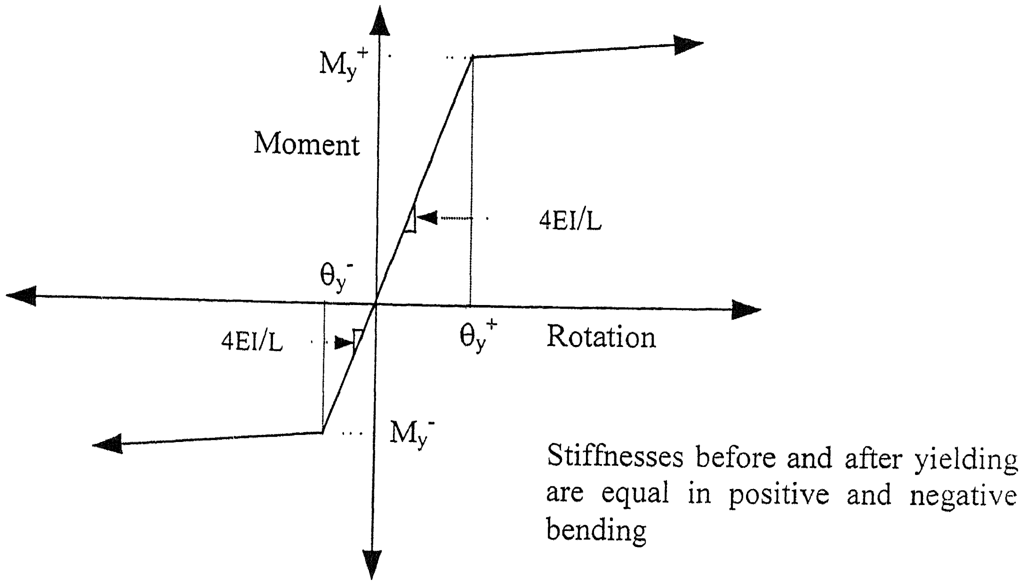


Fig. 2.2 : Moment-Rotation Relationship used in Element 2

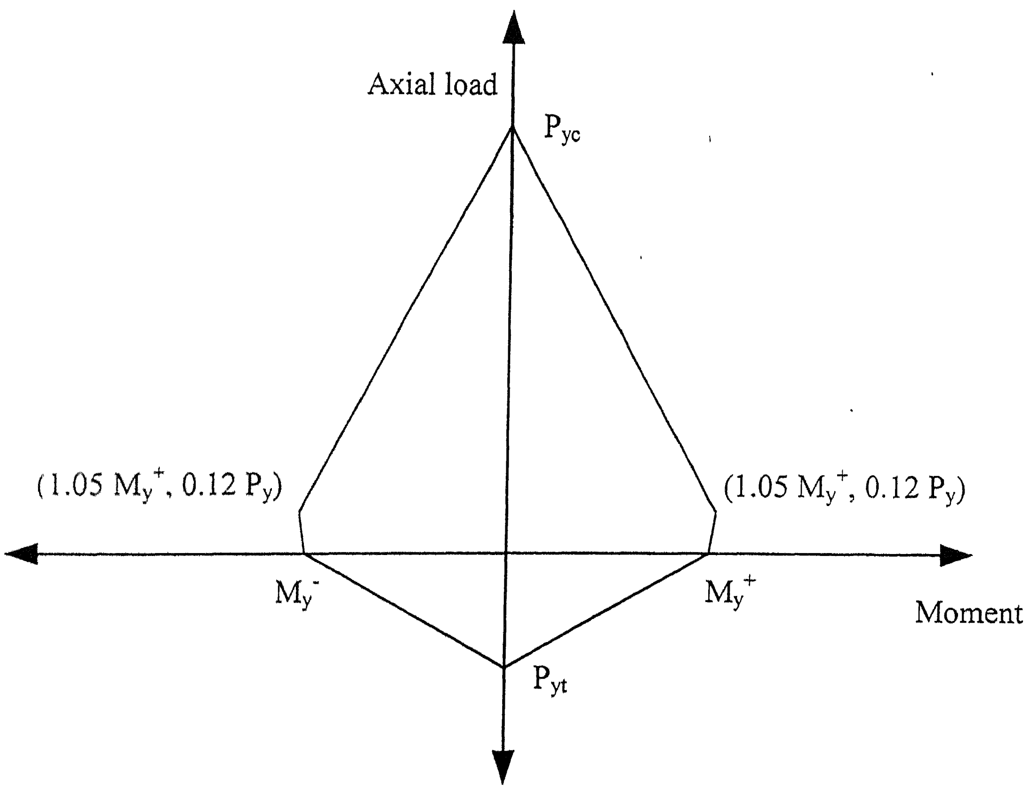


Fig. 2.3 : P-M Interaction Adopted in this Study

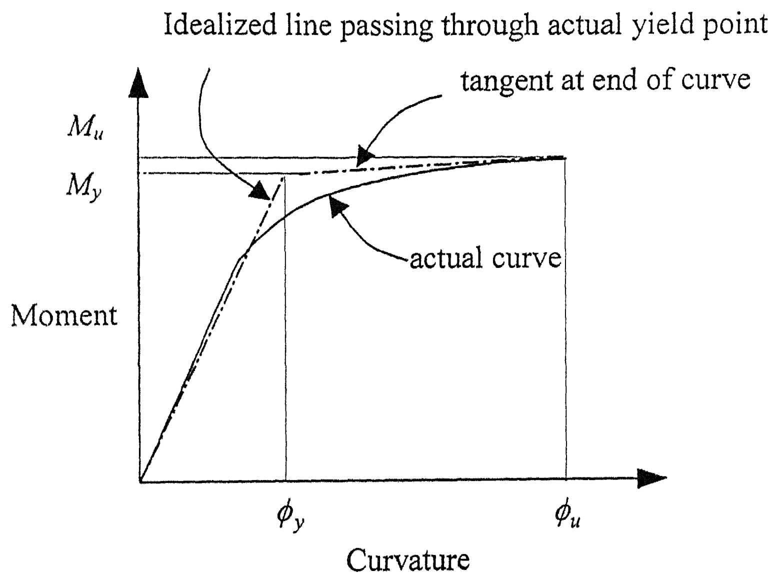
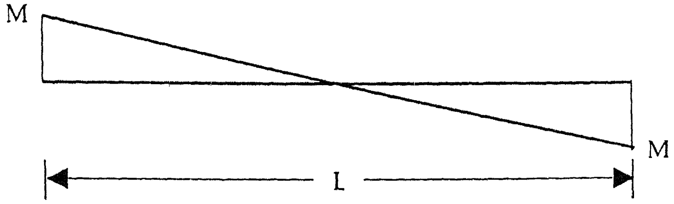
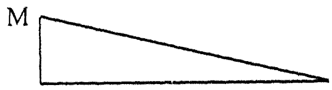


Fig. 2.4 : Idealization of Moment-Curvature Relationship

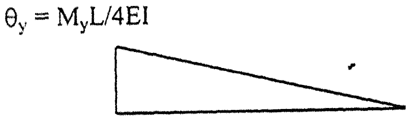


a) Bending Moment Diagram Due to Lateral Loads

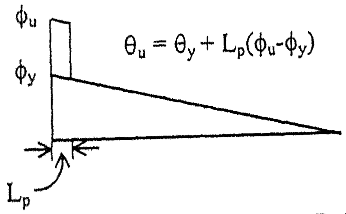
b) Equivalent Cantilever Beam



c) Bending Moment Diagram Due to Equivalent Cantilever Beam



d) Rotation at Yield Point



e) Rotation at Ultimate Point

Fig. 2.5 : Calculation of M-θ Relationship

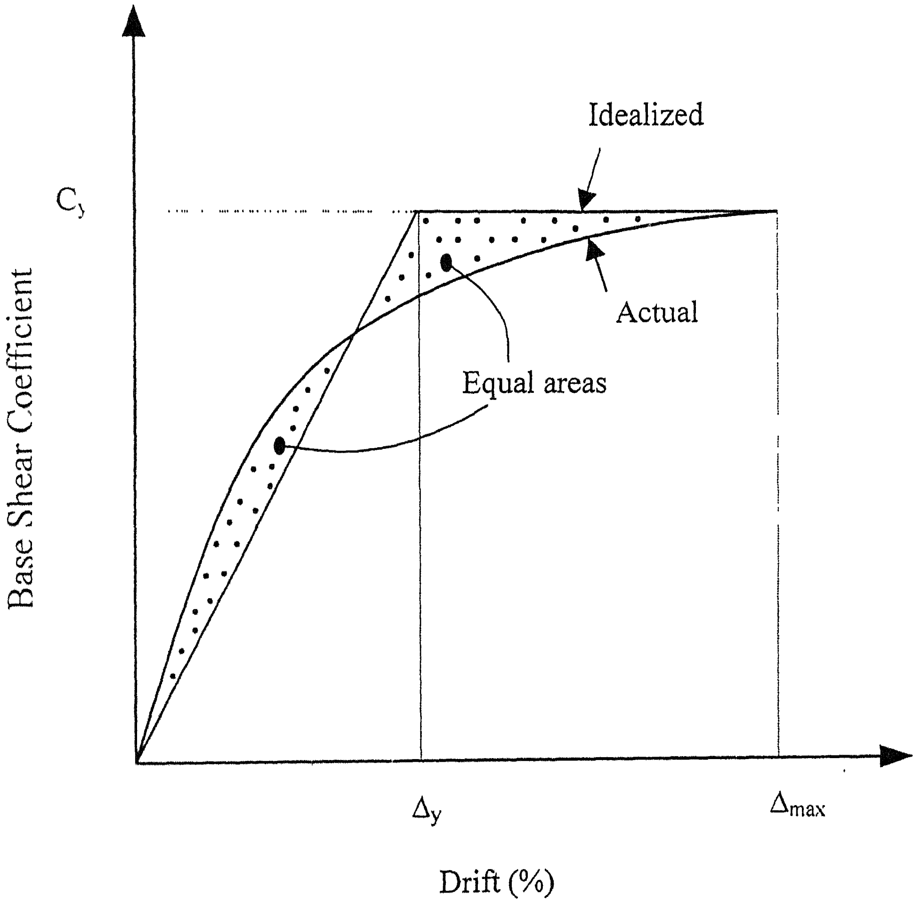


Fig. 2.6 : The Idealized Elastic-Perfectly Plastic Global Structural Response

Chapter 3

Behavior of a Typical Flat Slab Building

In this chapter a typical flat slab building is designed as per the IS code to represent the existing flat slab buildings and the nonlinear push-over analysis is carried out. The behavior of this building is compared with that of the conventional beam-column framing. The improvement in behavior due to continuous, column strip bottom reinforcement is also studied.

3.1 Design of the Building

A typical building with four stories and three bays in each direction is considered in the present study (Fig. 3.1). It represents existing flat slab buildings, which have not been detailed for earthquake loads but have been designed for wind loads. The bay width is 6 m in one direction and 5 m in another. The height of the first-storey is 4.4 m and that of the other storeys is 3.2 m.

For the purpose of design, linear structural analysis of the building is carried out by the computer program SAP2000 (Structural Analysis Program, Version 6.11). The slab of the building is assumed to be acting like a rigid floor diaphragm. The building is designed as per IS:456-1978 using limit state method of design. The load combinations used are as follows.

1.5 DL + 1.5 LL

1.2 DL + 1.2 LL ± 1.2 WL

1.5 DL ± 1.5 WL

0.9 DL ± 1.5 WL

For the purpose of wind load calculations, the structure is assumed to be situated in Bombay. The wind loads are calculated as per the IS:875 (Part 3)–1987:

$$V_z = V_b\, k_1\, k_2\, k_3 \qquad \ldots\ldots\ 3.1$$

where V_z = design wind speed at any height z in m/s, V_b = basic wind speed in m/s (= 44 m/s for building in Bombay), k_1 = the probability factor or risk coefficient (= 1.0 for all general buildings and structures), k_2 = terrain, height and structure size factor, and (= 1 for open terrain with well scattered obstructions and structures with components such as cladding, glazing, roofing, etc, having maximum dimension less than 20 m), and k_3 = topography factor (= 1.0). The design wind pressure (p_z) is calculated as follows:

$$p_z = 0.6\ V_z^{\,2} \qquad \ldots\ldots\ 3.2$$

From the above equations, the total design base shear on the building was obtained as 243.4 kN for a design wind pressure of 1.162 kN/m^2 acting along X-direction. This is 2.8% of the weight of the building. This force is distributed equally to all the four frames. The total design base moment due to this load is 1,704 kN-m. For the same building, the design seismic base shear and base moment will be 221.2 kN and 2,525 kN-m, respectively, in seismic zone II, and 442.4 kN and 5,052 kN-m in seismic zone III as per IS:1893-1984. The lateral load distribution along the height of the structure due to wind load is uniform, while that of the seismic loads is parabolic with larger forces at top. Hence, for the same value of base shear for wind and seismic loads, the base moment is larger for seismic loads.

The materials used are M 20 grade concrete and Fe 415 grade steel. Design and detailing has been done as per usual partial safety factors specified in IS:456-1978. Since the concrete strength increases with its age, it is assumed that the existing flat slab building is at least one year old. The design codes generally allow a factor of 1.2 to 1.25 on concrete strength as age factor for one-year-old concrete. Therefore, the characteristic

cube strength of M20 concrete is taken as 25 N/mm^2 for evaluating moment-curvature relations of elements needed for the push-over analysis and no partial safety factors on material strengths are assumed. A factor of 0.8 is used for converting cube strength to cylinder strength.

The thickness of the slab is 200 mm and no shear reinforcement is provided in the slabs. The columns are 300 mm square in section and are provided with ordinary detailing and no special confinement reinforcement is provided as is done in case of seismic detailing. After the design of the building, the slab-column connections are checked for punching shear failure by the eccentric shear stress model. The reinforcement details of all the elements of the frame are given in Table 3.1. The sizes of drop panels and column heads are shown in Fig. 3.2.

3.2 Procedure of Analysis

The building considered was designed as per the Indian code provisions so as to represent the existing flat slab buildings in India. The three-dimensional structure is idealized as a two-dimensional frame. The frame consists of original columns and equivalent beams defined by the slab depth and a portion of the slab width. Only a typical interior frame of the flat slab building is considered in this study. Displacement-controlled push-over analysis is carried out on the interior frame till a total drift of 2.5 % is reached or when any of the columns fail. This drift limit is generally considered adequate for this type of analysis. For this purpose the computer program "Structural Non-linear Analysis Program" (SNAP-2DX) is used.

While carrying out the push-over analysis the frames are applied full dead load and half the design live load (i.e., DL + 0.5 LL). The design live load on the floors is taken as 4 kN/m^2 except on the roof where it is 1.5 kN/m^2.

The moment-curvature (M-ϕ) relationship of the individual elements is evaluated by the program developed by Mandal [1993]. The slab-column connections were checked for the punching shear failure by eccentric shear stress method.

3.3 Details of Elements and Framing

The column heads are modeled as separate column elements and the drop panels are modeled as separate beam elements. The drop panel element extends from the center of slab-column connection to the end of the drop panel. The column head extends from the center of the slab-column connection to the end of the column head. The nodes at the base of the first storey columns are fixed. All nodes, other than those at the floors and those at base of ground storey columns, have 3 degrees of freedom. The nodes at the floors are forced to have equal translation in horizontal direction. There are a total of 32 column elements (8 for each floor level) and 36 beam elements (9 for each floor level), and the total number of degrees of freedom in the frame is 156.

Dovich and Wight [1994] proposed effective slab widths for both strength and stiffness as shown in Fig. 1.8. These effective slab widths are used in this study by taking the average of these values, so that a single effective slab width is used for all calculations. The effective width of slab for stiffness and strength are $l_2/3$ and $l_2/2$, respectively. Hence, an average value of $l_2/2.5$ is used for effective width of slab element. The effective width of drop panel is taken as its full width. The cross-sectional dimensions of the column head elements are taken by averaging their cross-section throughout the length.

3.4 Force-Displacement Relationship of Ordinary Flat Slab Frame

The push-over analysis of an interior frame is carried out in the longitudinal direction of the building till a total drift of 2.5%. The total drift is defined as the ratio of roof displacement to the height of the structure. The base shear coefficient versus the total drift (termed as 'drift' from here on) relationship is shown in Fig. 3.3. The sequence of plastic hinges is given in Fig. 3.4. The force-displacement relationship is linear upto 0.5% drift. The drop panel elements started yielding under sagging bending moment after 0.5% drift, and this causes softening of the force-displacement curve. This process continued till a drift of 1.13% is reached when all the drop panel elements have undergone yielding under sagging moment. The formation of early hinging at a drift of 0.5% is due to the absence (or inadequate) bottom reinforcement. In first-storey and second-storey slabs there was a small amount of continuous bottom reinforcement because of the load combination (0.9 DL + 1.5 WL), while the other floors have no continuous bottom reinforcement.

The bending moment diagram due to gravity loads and that due to lateral loads is shown in Fig. 3.5. The beam hinging in sagging or hogging occurs when the bending moment due to gravity plus lateral loads exceeds the moment capacity in sagging or hogging, respectively. Since the sagging reinforcement near supports is either zero or very small, there is hardly any sagging moment capacity near the support. Consequently, the hinging in sagging occurs soon after the bending moment due to lateral load exceeds the bending moment due to gravity load.

Due to the early hinging of drop panel elements, the damage starts at very low drift itself and continues for a long range. The overstrength when the first yield takes place is only 1.64 (Table 3.2), which is a rather low value. Hence, significant damage may take place in flat slab buildings, even for low intensity of ground shaking.

After the yielding of all the drop panel elements under sagging moment (at 1.13% drift), there was no further yielding till a drift of 1.8%. This is when the exterior right-end first-storey column yielded at its base. After this, the first-storey drop panel elements yielded under hogging moment. This is followed by yielding of other first-storey columns at the base, greatly reducing the stiffness.

At a drift of 1.99% the first-storey interior columns failed at their base by reaching their ultimate rotation capacities. The right-end and left-end first-storey exterior columns failed (i.e., reached their ultimate moment capacities) at drifts of 2.02% and 2.07%, respectively. In a drift range of 1.89% to 2.26% the second-storey drop panel elements yielded under hogging moment. However, as discussed in subsequent section, the frame may be considered to have reached failure point at 1.99% drift and the analysis beyond this drift cannot be relied upon. At this stage, the base shear coefficient is 0.109 and the overstrength is 3.89 (Table 3.2).

The nature of the force-displacement relationship is such that there is no well defined yield point, and the structure has hardly any ductility.

3.5 Limitations of the Present Analysis

The 'element 2' in SNAP-2DX used in push-over analysis assumes same stiffness of M-θ curve for both hogging and sagging bending moments, but allows different yield moments in hogging and sagging. More over, the program does not consider the M-θ curve of elements to be having a failure point. Beyond yield, the M-θ curve goes on continuously with a different, reduced stiffness specified by the user (Fig. 2.2).

In actual behavior, the elements may have a limited rotation capacity. However, because of the limitation of the program, the possibility of a node reaching the ultimate rotation capacity (θ_u) in any elements could not be automatically checked. The column

failures can lead to significant reduction in overall stiffness of the structure and may cause sudden collapse of the overall structure under earthquake condition. This is especially true in case of columns without confinement. Hence, the drift at which the column failure occurs when 'θ_u' is reached in columns is determined externally by another program. However, beam failures may only have localized effects. Hence, the possibility of θ_u exceeding in beam elements has not been checked.

Even after the failure of an element, the push-over analysis continues to assume its participation. Thus, any subsequent failures after the failure of first element are determined based on participation of elements failed previously. Meanwhile, in the actual structure because of first column failure, which will be generally in lower storeys, the stiffness of the structure greatly reduces. As a result, the base shear versus roof displacement relationship of actual structure starts dropping down. With more columns failing, this drop enhances further (Fig. 3.3). Considering this fact the maximum base shear capacity of the structure is taken as the base shear at the occurrence of first column failure or when 2.5% drift is reached whichever happens earlier. The same definition of maximum base shear capacity is applied for the subsequent analyses also.

3.6 Comparison of Flat Slab Frame with Beam-Column Frames

It is of interest to compare the seismic response of typical existing flat slab building with that of the beam-column frames. For this purpose, the results of push-over analysis on beam-column interior frames by Jain and Navin [1995] are used. These frames do not match exactly with the frame in this study in terms of number of storey, number of bays and design base shear. Hence, those frames which match as closely as possible are chosen for the purpose of comparison. Thus, a three-storey four-bay frame designed for a base shear coefficient of 0.04 (seismic zone III) and a six-storey four-bay

frame designed for a base shear coefficient of 0.03 (also in seismic zone III) are used. The flat slab building was designed for a base shear coefficient of 0.028. Although this is not an exact comparison, it will give a useful qualitative information.

Fig. 3.6 shows the force-displacement relationship for the flat slab frame and the beam-column frames. The important parameters for these buildings are given in Table 3.3. The push-over analysis of beam-column frames was carried out upto a total drift of 2 5%. It is seen that the beam-column frames possess considerably more lateral stiffness than the flat slab frame (about 70%). The drop panel elements started yielding in sagging moment at 0.55% drift itself. At this stage the overstrength was as low as 1.61.

The beam-column frames sustained a drift level of 2.5% and will sustain considerably more drift if still loaded. On the other hand, the first-storey interior columns of flat slab frame failed at 2% drift. Clearly, the flat slab building has low drift capacity and hardly any ductility. The inadequate ductility and deficient drift capacity associated with first-storey column failures can lead to a hazardous overall failure of the flat slab buildings under earthquake excitation.

Although the ductility and drift capacity of flat slab building is significantly less than that for the beam-column framing, the overstrength of flat slab building (=3.89) is quite comparable to that in the beam-column frames (=4.13 and 4.33). Moreover, the flat slab building was designed for wind load, which is uniformly distributed along the height of the building, while the beam-column frames were designed for seismic force distribution as per IS:1893-1984. Had the flat slab building been designed for the same force distribution as that of the beam-column frames, the resulting overstrength might have been somewhat larger than the one obtained here.

3.7 Response with Continuous Bottom Reinforcement

IS:456–1978 does not have stringent requirements for the purpose of continuous bottom reinforcement in flat slabs. The bottom reinforcement can be curtailed near the supports except where required by the analysis. As per the ACI 318-1995, the entire column strip bottom reinforcement should be continuous. This gives the slab some residual ability to span to the adjacent supports if a single support gets damaged. Apart from this, ACI requires that at least two of the column strip bottom bars in each direction shall pass through the column core and shall be anchored at exterior supports. The two continuous column strip bottom bars through the column are termed 'integrity steel', and are provided to give the slab some residual capacity following a single punching shear failure. This provision was incorporated on account of the research by Mitchell and Cook [1984] to arrest progressive collapse.

The building of present study is modified to have continuous column strip bottom reinforcement as per ACI requirement and push-over analysis is carried out. The response obtained is compared with that of the ordinary flat slab frame (Fig. 3.7 and Table 3.4).

Provision of continuous bottom reinforcement leads to significant improvements in the lateral behavior as seen through the force-displacement relationship. The first yield point now occurs at 0.86% drift (as against 0.5% drift earlier). Thus, overstrength at first yield is 2.82 (1.64 earlier). Moreover, the force-displacement curve does not show significant softening on account of sagging moment hinges and the structure is able to retain its lateral stiffness. The sagging hinges occurred in the drift range 0.86% to 1.29%. This was followed by hinging of first storey columns at base at drift of 1.46%. Both the first storey interior columns failed at 1.57% drift.

Frame with continuous bottom reinforcement could sustain loading upto base shear coefficient of 0.12 giving an overstrength of 4.29; an improvement of about 10%.

On the other hand, drift capacity reduced by about 20% (to 1.57%). Due to improved behavior of frame under sagging moment, the frame stiffness improved and hence, the columns underwent yielding and failure at lower drift levels. Even though the drift capacity reduced somewhat, the response of frame, under earthquake excitation is expected to be significantly improved on account of the continuous bottom reinforcement. This is so in view of a) delayed occurrence of first yield, b) absence of significant stiffness reduction due to sagging hinges, and c) improved overstrength. Clearly, the ACI provisions on 'integrity steel' and continuous bottom reinforcement help improve the seismic response. However, the frame still possesses practically no ductility, and such frames still will be quite deficient in seismic response.

3.8 Statement of the Problem

The existing flat slab buildings have low stiffness and low drift capacity than the conventional beam-column frames, while the overstrength is quite comparable. The force-displacement relation of flat slab structure hardly shows any ductility. The drop panel elements start yielding under sagging at very low drift due to the insufficient bottom reinforcement. When the existing buildings are provided with continuous column strip bottom reinforcement, the yielding of drop panels started at greater drift and was much less compared to that of the original building. Even with this way, the problems associated with insufficient drift capacity and absence of ductility remain and such buildings require seismic retrofitting.

Table 3.1 (a) : Details of Column Elements

Column	Length (mm)	Size (mm)	Reinforcement (%)
C11	3575	300 X 300	2.53
C12	3575	300 X 300	5.07
C21	2375	300 X 300	2.53
C22	2375	300 X 300	4.22
C31	2375	300 X 300	2.11
C32	2375	300 X 300	1.69
C41	2375	300 X 300	2.11
C42	2375	300 X 300	1.69
CH11	825	1000 X 1000	2.53
CH12	825	1000 X 1000	5.07
CH21	825	1000 X 1000	2.53
CH22	825	1000 X 1000	4.22
CH31	825	1000 X 1000	2.11
CH32	825	1000 X 1000	1.69
CH41	825	1000 X 1000	2.11
CH42	825	1000 X 1000	1.69

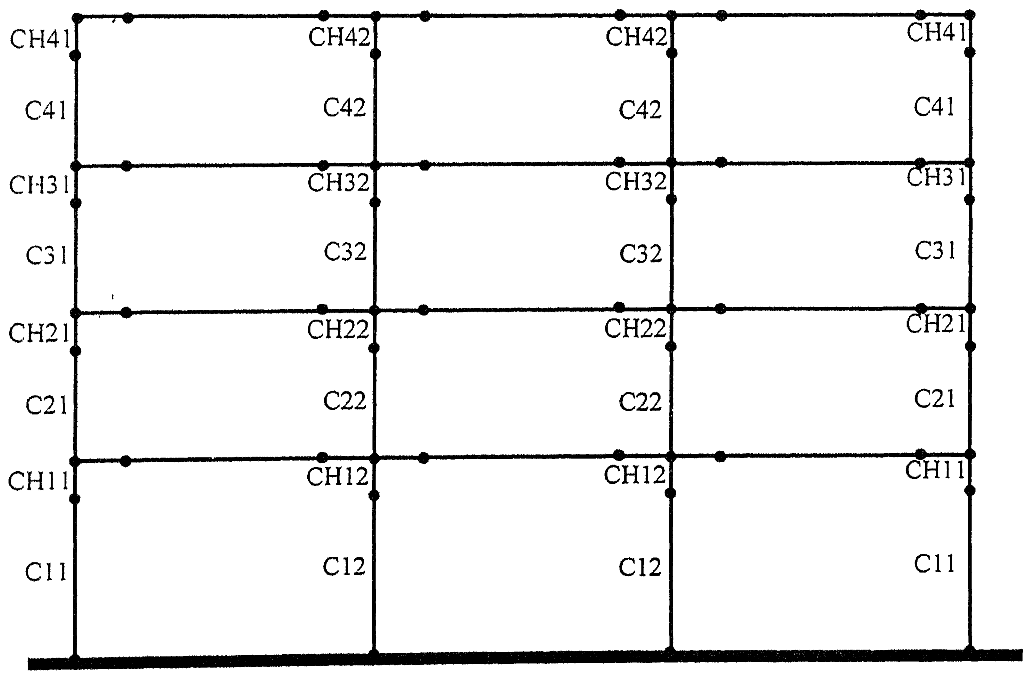


Table 3.1 (b) : Details of Beam Elements

Beam	Length (mm)	Size (mm)	Reinforcement at Element Ends (%)	
			Top	Bottom
B1	4000	2000 X 200	0.481	0.113
B2	4000	2000 X 200	0.452	0.113
B3	4000	2000 X 200	0.368	Zero
B4	4000	2000 X 200	0.339	Zero
DP1	1000	1700 X 250	0.396	0.113
DP2	1000	1700 X 250	0.311	0.113
DP3	1000	1700 X 250	0.311	Zero

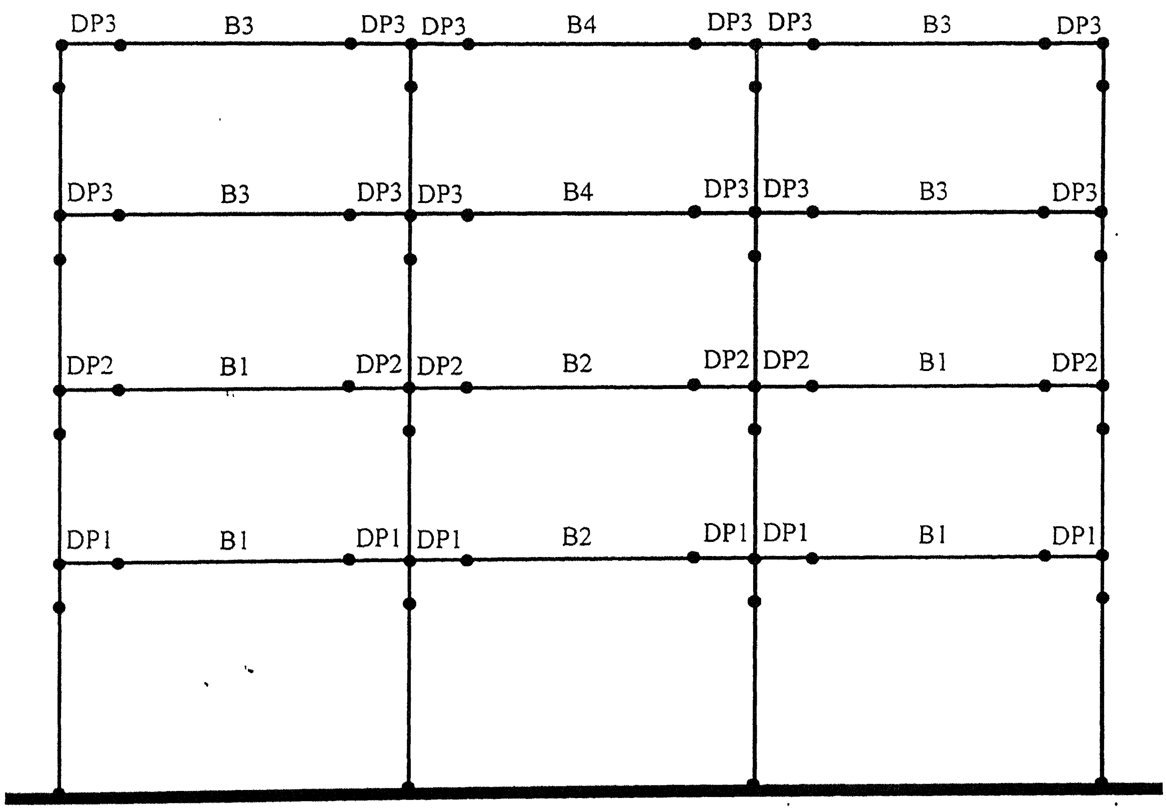


Table 3.2 : Response of Ordinary Flat Slab Frame

Property			Value
Initial Stiffness = Base shear at first yield /Roof Displacement (Base Shear Coefficient/Drift (%))			1.43 kN/mm (0.093)
Overstrength at First Yield (C_f/C_w)	C_f	0.046	1.64
	C_w	0.028	
Overstrength (C_y/C_w)	C_y	0.109	3.89
	C_w	0.028	

Table 3.3 : Comparison of Ordinary Flat Slab Frame and Beam-Column Frames

Type of Framing	Ordinary Flat Slab Frame	Three-Storey Four-Bay Beam-Column Frame [Jain and Navin, 1995]	Six-Storey Four-Bay Beam-Column Frame [Jain and Navin, 1995]
Initial Stiffness = Base Shear Coefficient/Drift (%)	0.093	0.16	0.16
C_w	0.028	0.04	0.03
C_f	0.046	0.13	0.07
C_y	0.109	0.173	0.124
Δ_{max}	----	>2.50%	>2.50%
Δ_y	----	1.15%	1.00%
Overstrength $\Omega = C_y/C_w$	3.89	4.35	4.13
Ductility $\mu = \Delta_{max}/\Delta_y$	----	> 2.17	> 2.55

Table 3.4 : Comparison With and Without Continuous Bottom Reinforcement

Building Type	C_w	C_f	C_y	Overstrength at First Yield $\Omega_f = C_f/C_w$	Overstrength $\Omega = C_y/C_w$	Drift Capacity (%)
Without Continuous Bottom Reinforcement	0.028	0.046	0.109	1.64	3.89	1.99
With Continuous Bottom Reinforcement	0.028	0.079	0.120	2.82	4.29	1.57

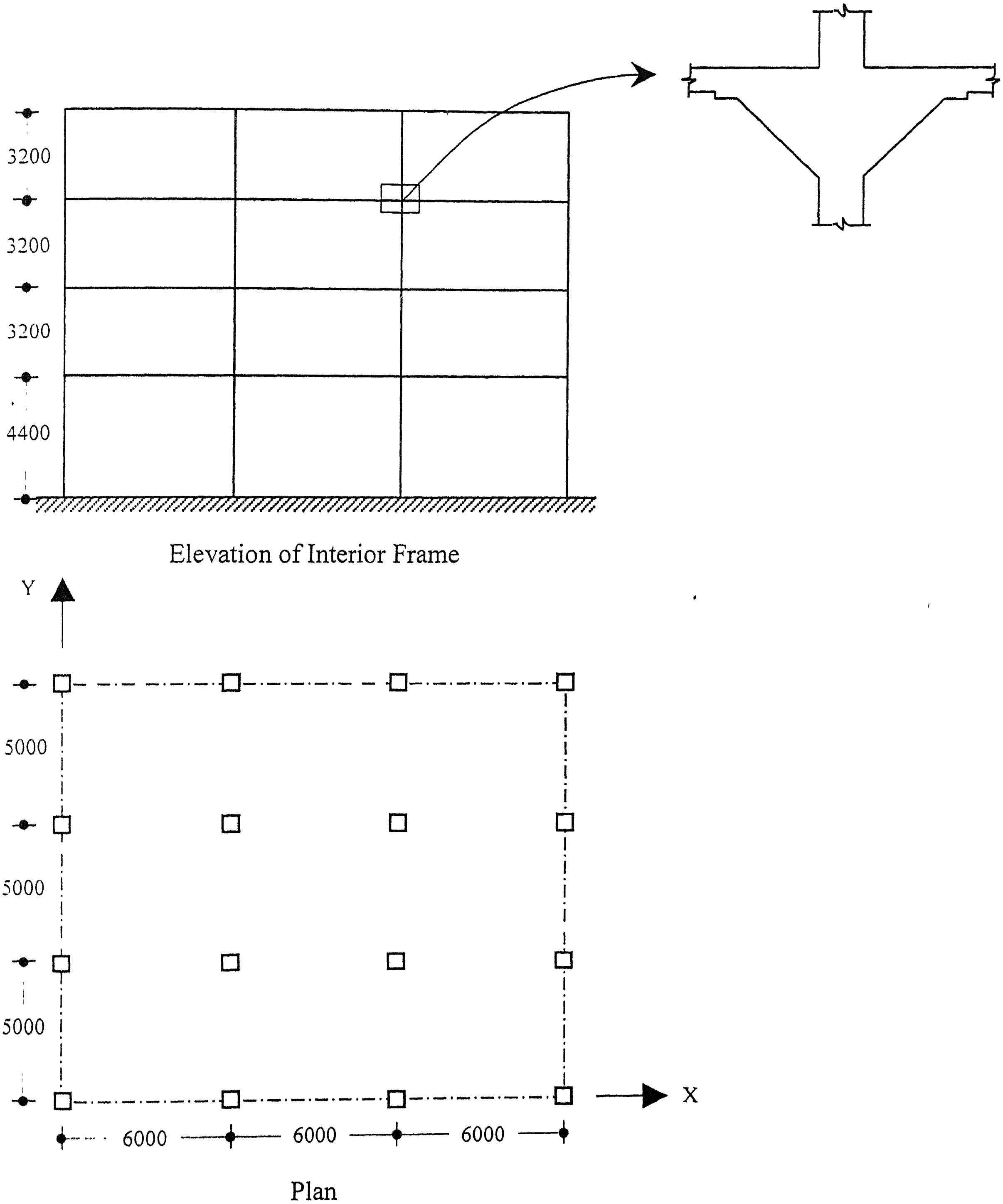


Fig. 3.1 : Flat Slab Building

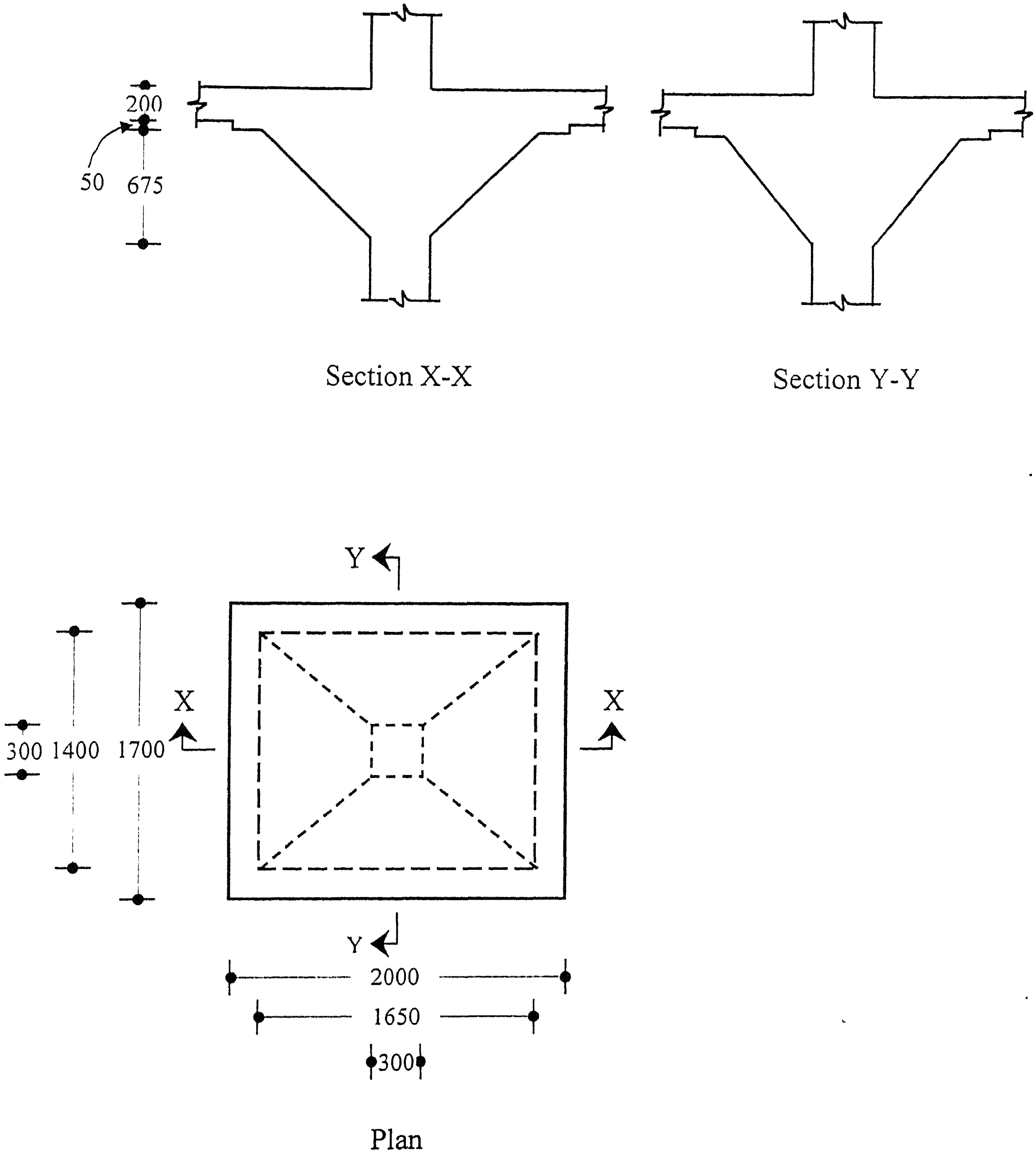


Fig. 3.2 : Dimensions of Slab-Column Connection

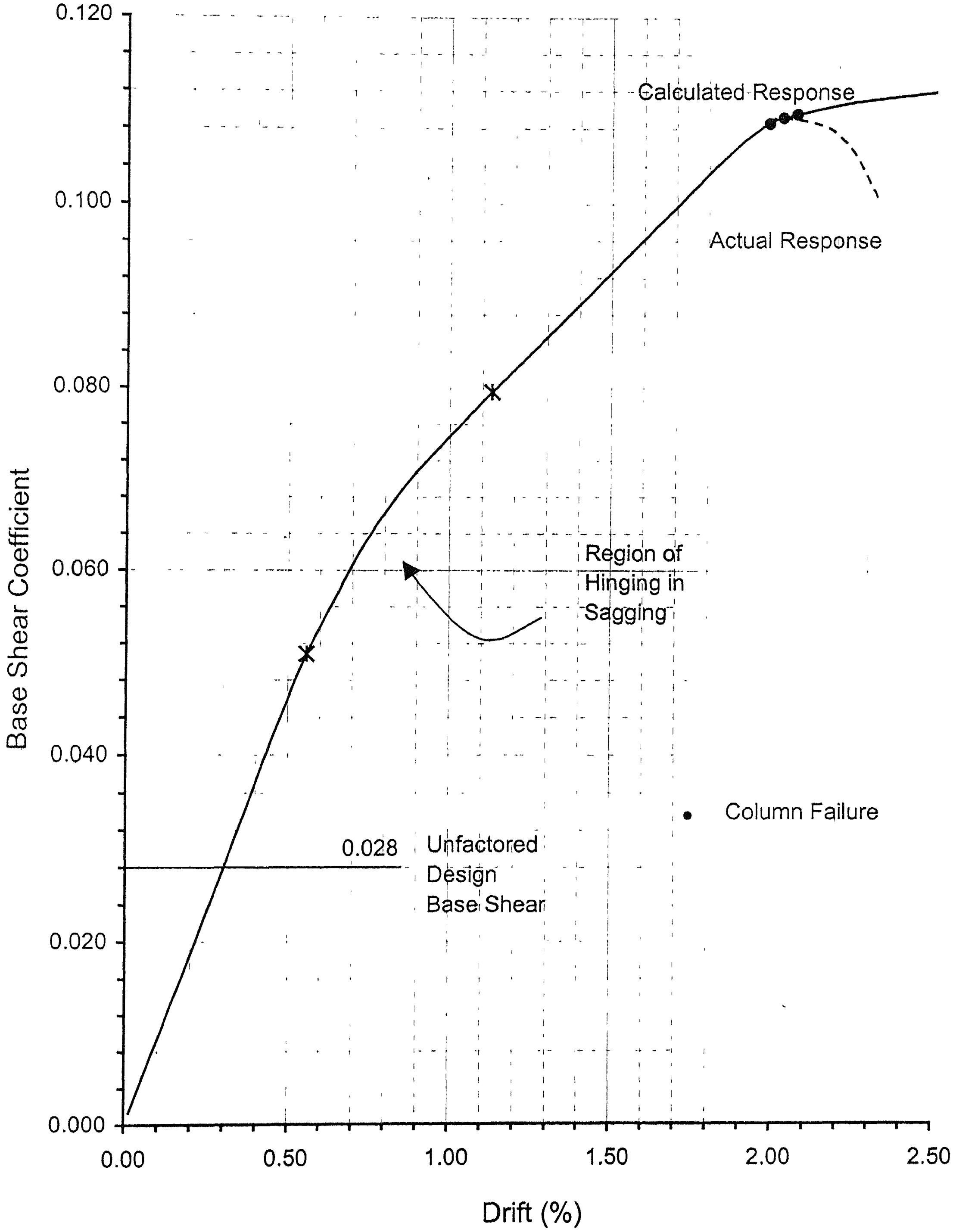
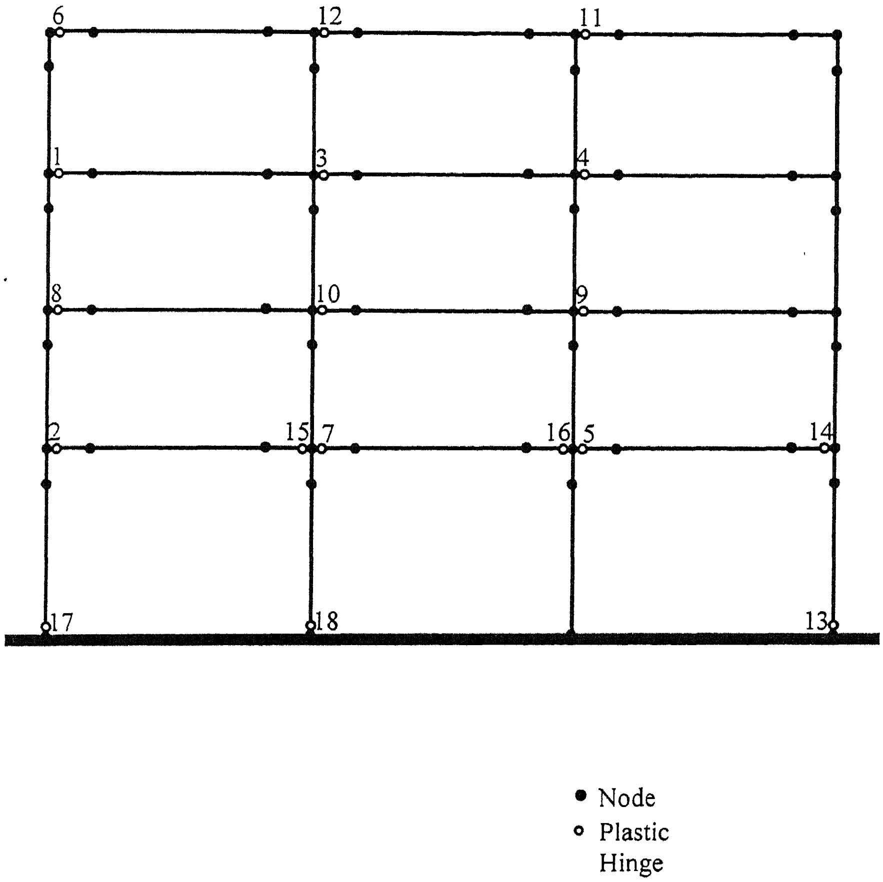


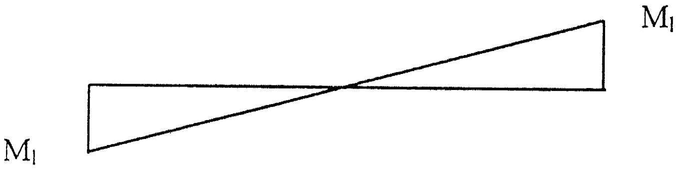
Fig. 3.3 : Force-Displacement Relationship of the Flat Slab Frame



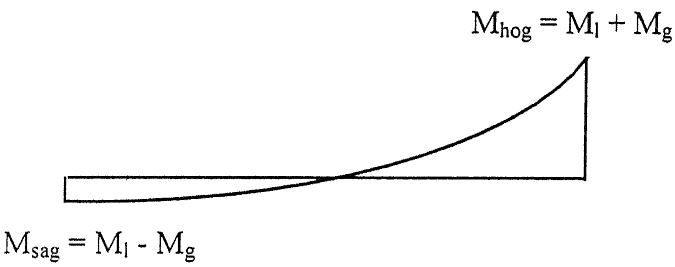
Plastic Hinges Sequence	Drift (%)
1	0.500
2	0.557
3	0.557
4	0 557
5	0.714
6	0.714
7	0.743
8	0.786
9	0.857
10	0.900
11	1.114
12	1.129
13	1.800
14	1.886
15	1.914
16	1.929
17	1.943
18	1.957

Fig. 3.4 : Sequence of Plastic Hinges in Flat Slab Frame

a) Bending moment due to gravity loads



b) Bending Moment due to lateral loads



<u>A hinge forms if</u>

a) $M_{sag} > M_{ysag}$

b) $M_{hog} > M_{yhog}$

ling Moment due to Combined Gravity and Lateral Loads

: Formation of Plastic Hinges in Beam Elements

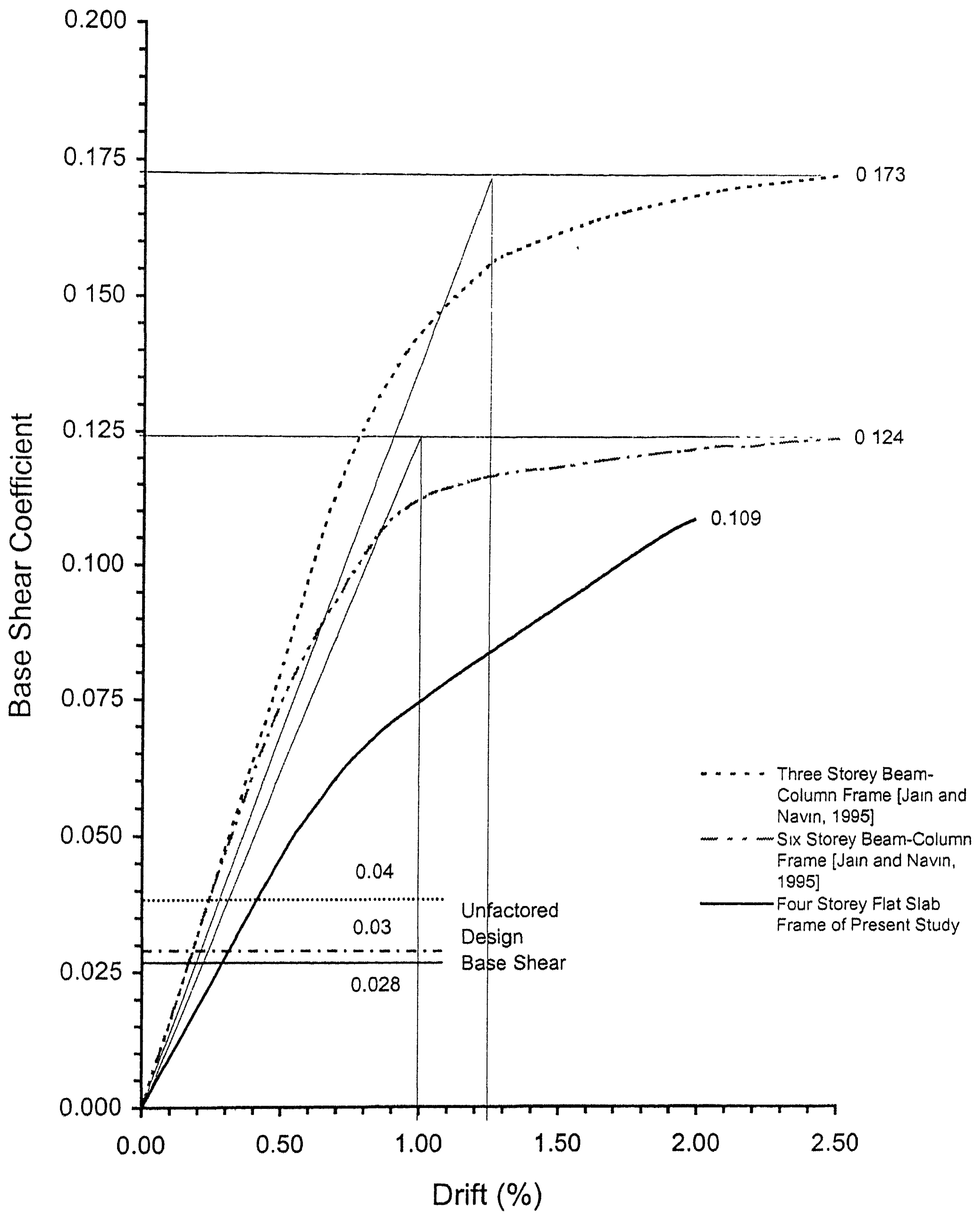


Fig. 3.6 : Comparision of Ordinary Flat Slab Frame with Beam Column Frames

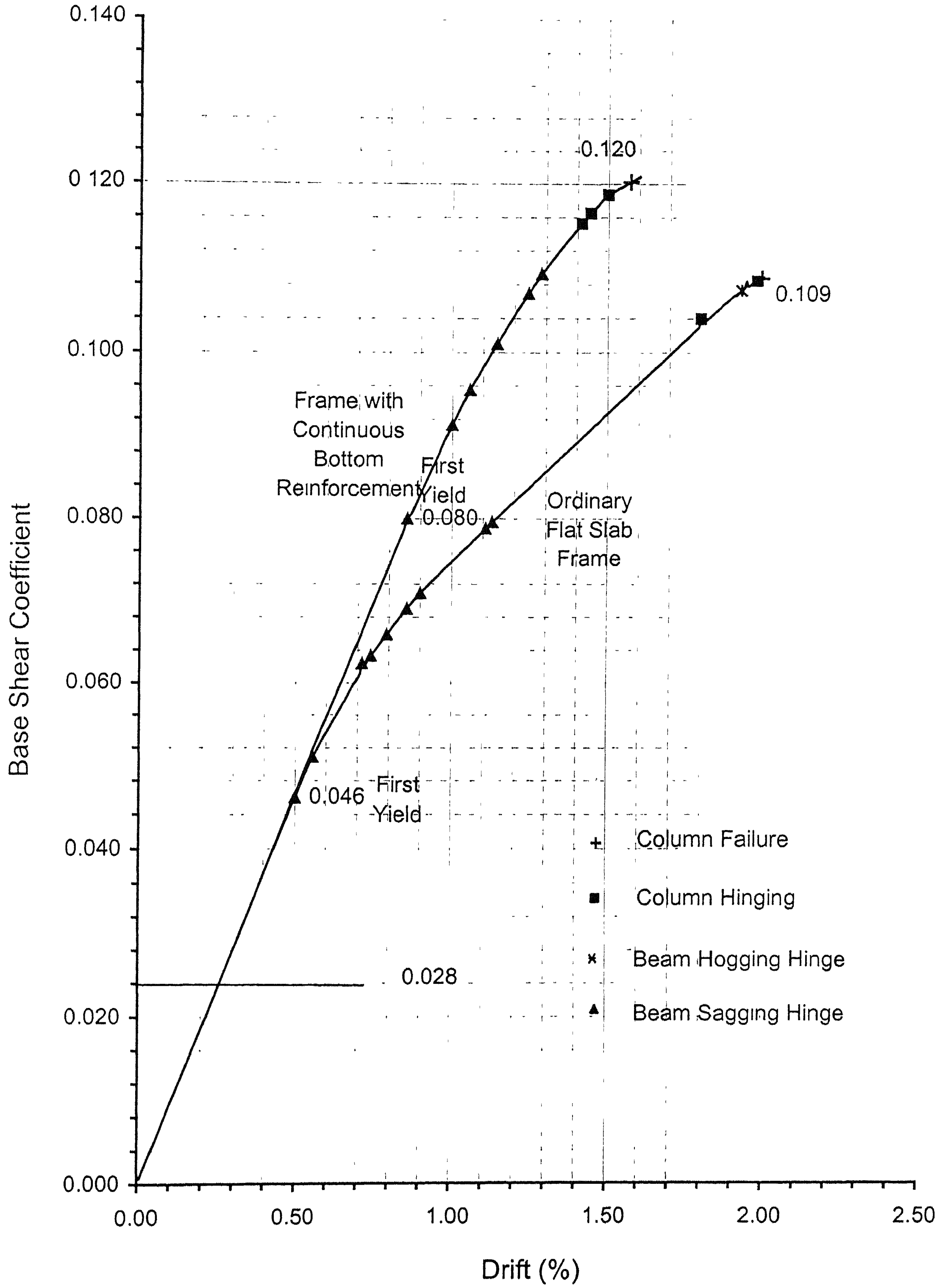


Fig. 3.7 : Comparison With Continuous Column Strip Bottom Reinforcement

Chapter 4

Behavior of Retrofitted Flat Slab Buildings

4.1 Introduction

Most of the existing flat slab buildings were designed only for gravity loading and sometimes for wind loading also. As seen in the previous chapter, these buildings have to be retrofitted for better seismic performance. Hence, in this chapter the response of retrofitted building is evaluated by push-over analysis. Retrofitting by column jacketing and/or by addition of extra beam to the slab along the column lines are considered. The retrofitting configurations studied here include a) retrofitting by column jacketing only, b) retrofitting by providing additional beams only, and c) retrofitting by both column jacketing and by additional beams. It is of interest to see how the retrofitting of different storeys would change the behavior of the flat slab buildings. Hence, in each of these three schemes, analysis is carried out assuming retrofitting of first storey only, first two storeys, first three storeys, and of all the storeys.

4.2 Retrofitting with Column Jacketing

4.2.1 Details of Column Jacketing

The existing columns of the flat slab building are retrofitted to increase their dimension from 300 mm x 300 mm to 500 mm x 500 mm. The increased column area is taken to the height till it flushes with the column head. In column jacketing, first the cover concrete is removed, and then the column is jacketed by extra concrete and steel reinforcement. The extra longitudinal reinforcement is taken through the slab by drilling holes (Fig. 4.1). These holes are then filled by grouting. Confinement reinforcement is

provided around the newly provided longitudinal reinforcement. The extra longitudinal reinforcement is equal to 1 % of the total column area. Confinement reinforcement is provided as per IS : 13920 – 1993. From this, the volume of transverse reinforcement came out to be 0.7% of the volume of column core (which is 10 mm dia. @ 100 mm c/c). The reinforcement details of retrofitted columns are given in Table 4.1.

4.2.2 Lateral Load Response

The force-displacement relations for the column retrofitting are shown in Fig. 4.2. The figure shows that the overstrength and the stiffness increase with number of storeys of column retrofitting. The increase in overstrength due to retrofitting of first storey, first two storeys, first three storeys, and all storeys of column retrofitting are 40.4%, 54.1%, 56.9%, 62.4%, respectively, with respect to the original structure (Table 4.3). Clearly maximum improvement in overstrength and initial stiffness is achieved by jacketing of ground storey columns. Jacketing of the upper storeys does not further improve the response in a significant way.

In all cases of column retrofitting, all the drop panel elements under sagging hinged before any other type of hinges formed. The first yielding started at around 0.31% drift in all these cases of retrofitting as against 0.5% in the original structure it was at 0.5%. The increased stiffness due to column retrofitting attracted more forces at lower drifts, and resulted in earlier yielding of drop panel elements. These extra forces are concentrated at retrofitted columns. Hence, the drop panels beside these columns yielded earlier than in original building. The drift range, in which the drop panel elements yielded under sagging, decreased slightly with increasing number of floors retrofitted. In all these cases, there was a long linear portion after drop panel yielding under sagging moment. This was in the range of about 0.71% to 1.79%.

The yielding of drop panel elements under hogging was also observed. In case of first storey retrofitting, it was observed in third storey only, while in all other cases of column retrofitting, it was observed in all storeys except the top storey. This is because the stiffer elements (i.e., retrofitted column elements) were attracting more forces. Hence, the drop panel elements, which were connected to these stiffer elements also attracted more forces, and have undergone yielding.

At higher drifts of above 2.14%, columns started yielding. The column hinging shifted to upper storeys, which were not retrofitted. In the original structure, column hinging was significant in lower storeys.

In general the column failures in unretrofitted buildings occur in lower storeys. But no column failure is observed in any of these four cases upto the 2.5% drift. This is due to the increase in ultimate rotation capacity of retrofitted columns. Moreover, the increased stiffness gained through retrofitting in these four cases reduce after drop panel yielding, and results in insufficient forces to cause failures in retrofitted columns.

4.3 Retrofitting with Addition of Beam

4.3.1 Details of Addition of Beam

To provide an extra beam, the bottom cover of slab is removed, and an extra beam is provided along with continuous bottom bars. These bars are taken through the slab-column connections by drilling holes through the column head. These holes are then filled by grouting. In this scheme, the flat slab building is provided with an extra thin beam along the column lines. This beam is projected 200 mm below the existing slab. The width of this beam is kept so that it flushes with the column head (i.e., 1146).

Different sections in hogging and sagging were assumed for the purpose of sectional properties of retrofitted beam elements as shown in Fig. 4.3. The reinforcement details of these sections are given in Table 4.2. The stiffness of these elements is then

taken as average of the hogging and sagging stiffness. This is due to the limitation of Element 2 of SNAP-2DX to allow only one value of stiffness in both sagging and hogging.

4.3.2 Lateral Load Response

The force-displacement relations for beam retrofitting are shown in Fig. 4.4 and important parameters summarized in Table 4.3. Beam retrofitting helps significantly improve stiffness. Moreover, the stiffness degradation due to sagging hinges is avoided. However, the structure remains without any worthwhile ductility. A significant increase in overstrength (37%) is observed with first storey beam retrofitting. No significant improvement in overstrength is achieved when more storeys were retrofitted. This is because, columns were predominantly the weaker elements after retrofitting by beam addition. On the other hand, initial stiffness improves significantly as more number of storeys are retrofitted by providing beams (37%, 68%, 118%, and 118%, respectively).

The increased stiffness due to retrofitting resulted in attracting more forces at small drifts. Consequently, the columns yielded and failed at lower drift than in case of the original building. Hence, the drift capacity reduced with increasing number of storeys of retrofitting. The failure of first column was observed at drifts of 2.14%, 1.59%, 1.06%, and 0.99% in case of first storey, first two storeys, first three storeys, and all storeys retrofitting, respectively. Thus, retrofitting of beams alone increases the stiffness and strength, but reduces the drift capacity (Table 4.3).

At a drift of around 0.36%, sagging hinges took place in beams at floors that were not retrofitted. Again, this happened at lower drift than in the original structure (0.50%) due to increased stiffness.

4.4 Retrofitting with Column Jacketing and Addition of Beam

4.4.1 Details of Retrofitting

In this scheme, in each retrofitted storey, both column jacketing and addition of floor beams are adopted simultaneously. The analysis is carried out for the four cases of retrofitting in different storeys as in the previous sections. The reinforcement and dimensional details of retrofitting are same as considered earlier.

4.4.2 Lateral Load Response

The force-displacement relations for these four cases are shown in Fig. 4.5 and important response quantities are summarized in Table 4.3.

One interesting aspect of this retrofitting scheme is that the damage shifted completely to the unretrofitted upper storeys.

As more number of storeys are retrofitted, there is a sigificant improvement in overstrength (38%, 55%, 162%, and 319%, respectively) and initial stiffness (78%, 168%, 331%, 470%, respectively). In fact, overstrength in the retrofitted building ranges from 5.39 to 16.32 depending on the number of storeys retrofitted. Overstrength at first yield decreases slightly (10% and 2%, respectively) for first two storey retrofitting, but improves significantly (69%, 37.9%) when three of four storeys are retrofitted. The frame response with respect to stiffness degradation due to sagging hinges improves as more storeys are retrofitted. In fact, a small amount of ductility (1.9) is seen in case of all storey retrofitting.

The drift capacity reduces somewhat (10% to 40%) when not all storeys are retrofitted. This is because partial retrofitting increases stiffness and the columns in unretrofitted storeys fail at lower drift. When all storeys are retrofitted, the drift capacity is slightly higher (5%) than in the original structure.

4.5 Comparison of Retrofitting Schemes with Number of Storeys of Retrofitting

The three schemes of retrofitting (i.e., column jacketing, addition of beams, column jacketing and addition of beams) are compared in Figs. 4.6 to 4.9.

In case of first storey retrofitting (Fig. 4.6), the improvement in overstrength obtained by all the three schemes is almost similar (37% to 40%). The stiffness is about the same for beam retrofitting alone and column retrofitting alone, but is higher for beam and column retrofitting.

For retrofitting of first two storeys also (Fig. 4.7), the overstrength from the three schemes is comparable (46% to 55%). The softening is significantly reduced by beam retrofitting, but the drift capacity is significantly less than that in case of column retrofitting (Table 4.3). Due to the column and beam retrofitting, stiffness increased even further and the drift capacity is less than that from beam retrofitting.

In case of retrofitting of first three storeys, increase in overstrengths due to beam retrofitting and column retrofitting are similar (40% and 57%, respectively), but much larger (162%) due to the column plus beam retrofitting. The drift capacity in beam retrofitting and beam plus the column retrofitting are significantly less than that in the original structure and in case of column retrofitting.

When all the storeys are retrofitted, improvement the overstrength due to beam retrofitting and column retrofitting are similar (42% and 62%, respectively), but much higher (319%) due to beam plus column retrofitting. The drift capacity of beam retrofitting is far less than that due to other retrofitting schemes and that of the original structure. The stiffness of beam plus column retrofitting is very high compared to the original structure and other two retrofitting schemes.

From Figs. 4.8 and 4.9, it is clear that increasing the number of storeys of retrofitting either by beam retrofitting alone or by column retrofitting alone does not increase the overstrength considerably. On the contrary the overstrength due to retrofitting by both column plus beam retrofitting was significantly higher than beam retrofitting alone and column retrofitting alone.

In all the above cases, it can be observed that the beam retrofitting gives more stiffness but less drift capacity than the column retrofitting. The less drift capacity is because of the unretrofitted column failures. This effect is predominant in all cases except when only ground storey was retrofitted. The drop panel yielding in sagging is similar to original building in case of column retrofitting, while it is limited to unretrofitted floors in beam retrofitting.

The beam plus column retrofitting gives higher strength and stiffness than the other two schemes. The damage is shifted to unretrofitted upper floors completely by this scheme. The increased stiffness of the structure attracted higher forces at less drift and the unretrofitted columns failed earlier resulting in less drift capacity than the original structure. When all storeys were retrofitted with beam and column retrofitting the drift capacity also improved considerably and was even higher than the original structure despite the significant increase in stiffness.

Table 4.1 : Details of Retrofitted Column Elements

Column	Length (mm)	Size (mm)	Longitudinal Reinforcement (%)
C15	3575	500 x 500	1.98
C16	3575	500 x 500	2.89
C25	2375	500 x 500	1.98
C26	2375	500 x 500	2.58
C35	2375	500 x 500	1.82
C36	2375	500 x 500	1.67
C45	2375	500 x 500	1.82
C46	2375	500 x 500	1.67
CH15	825	1400 x 1400	0.25
CH16	825	1400 x 1400	0.37
CH25	825	1400 x 1400	0.25
CH26	825	1400 x 1400	0.33
CH35	825	1400 x 1400	0.23
CH36	825	1400 x 1400	0.21
CH45	825	1400 x 1400	0.23
CH46	825	1400 x 1400	0.21

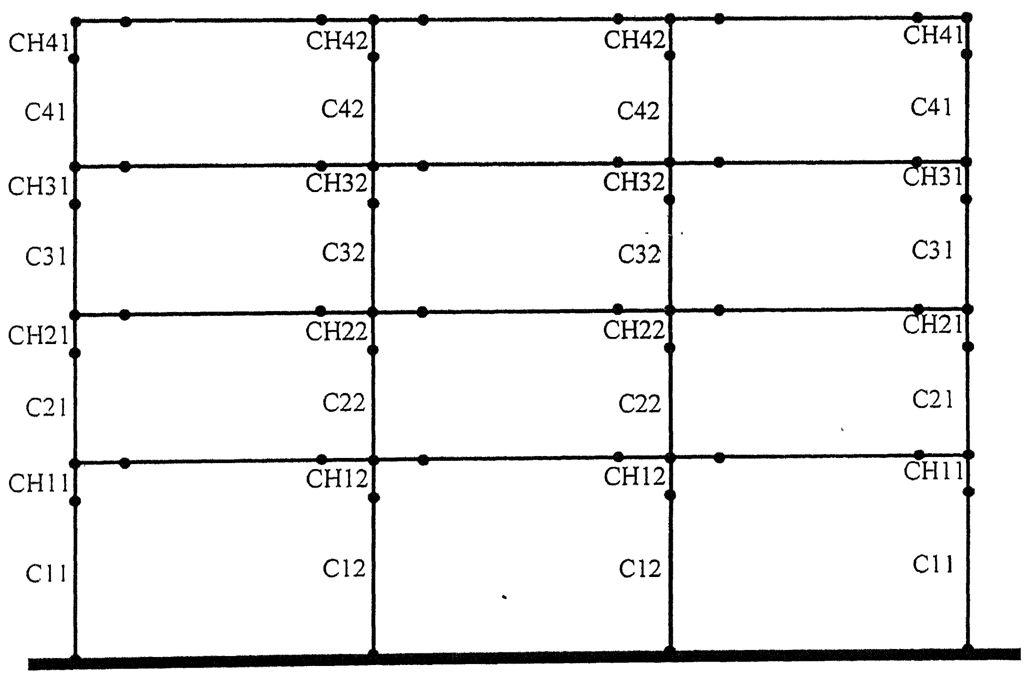


Table 4.2 : Details of Retrofitted Beam Elements

Beam	Length (mm)	Size (mm)		Reinforcement at Element Ends (%)	
		Hogging	Sagging	Top	Bottom
B1	4000	1146 x 400	2000 x 400	0.42	0.95
B2	4000	1146 x 400	2000 x 400	0.39	0.95
B3	4000	1146 x 400	2000 x 400	0.32	0.95
B4	4000	1146 x 400	2000 x 400	0.30	0.95
DP1	1000	1146 x 400	1700 x 400	0.34	0.95
DP2	1000	1146 x 400	1700 x 400	0.27	0.95
DP3	1000	1146 x 400	1700 x 400	0.25	0.95
DP4	1000	1146 x 400	1700 x 400	0.27	0.95

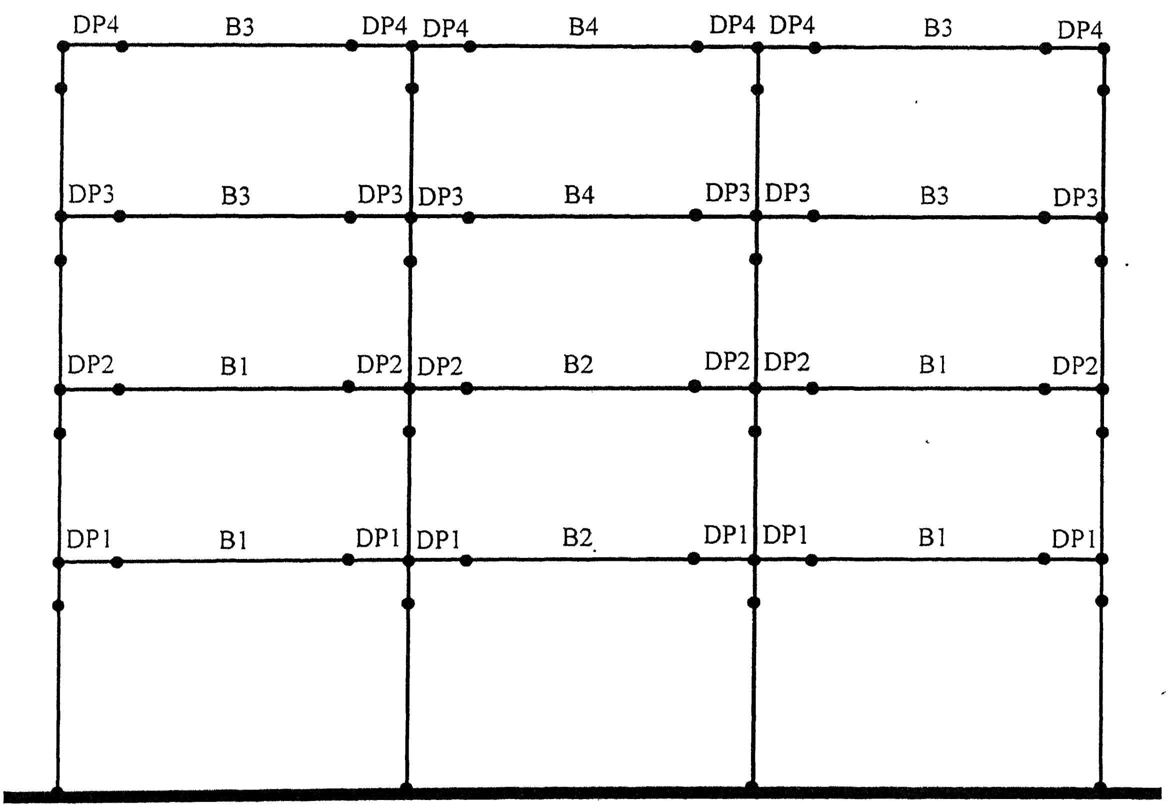


Table 4.3 : Comparison of Different Retrofitting Schemes with Original Building

	Drift Capacity (mm)			Overstrength $\Omega = C_y/C_w$			Overstrength at First Yield $\Omega_f = C_f/C_w$			Initial Stiffness = (Base Shear Coefficient / %Drift)		
Type of Retrofitting / No. of Storeys From Bottom Retrofitted	Column Retg.,	Beam Retg.,	Beam and Column Retg.,	Column Retg.,	Beam Retg.,	Beam and Column Retg.,	Column Retg.,	Beam Retg.,	Beam and Column Retg.,	Column Retg.,	Beam Retg.,	Beam and Column Retg.,
1	> 350 (>26%)	292 (5%)	250 (-10.1%)	5.46 (40.3%)	5.32 (36.8%)	5.39 (38.5%)	1.43 (-12.8%)	1.49 (-9.1%)	1.47 (-10.4%)	0.127 (41.1%)	0.123 (36.7)	0.16 (77.8)
2	> 350 (>26%)	222 (-20.1%)	182 (-34.5%)	6.00 (54.2%)	5.46 (40.3%)	6.04 (55.3%)	1.4 (-14.6%)	1.62 (-1.2%)	1.6 (-2.4%)	0.137 (52.2)	0.151 (67.8)	0.241 (167.8)
3	> 350 (>26%)	148 (-46.8%)	164 (-41%)	6.11 (57.1%)	5.46 (40.3)	10.18 (162%)	1.58 (-3.66%)	2.8 (70.7%)	2.77 (68.9%)	0.147 (63.3)	0.196 (117.8)	0.388 (331.1)
4	> 350 (>26%)	138 (-50.3%)	292 (5%)	6.32 (62.5%)	5.46 (40.3%)	16.32 (319%)	1.76 (7.3%)	4.57 (178%)	7.86 (379.3%)	0.155 (72.2)	0.196 (117.8)	0.513 (470)
Original Building	278			3.89			1.64			0.09		

The numbers in bracket indicate percentage increase with respect to the original structure.

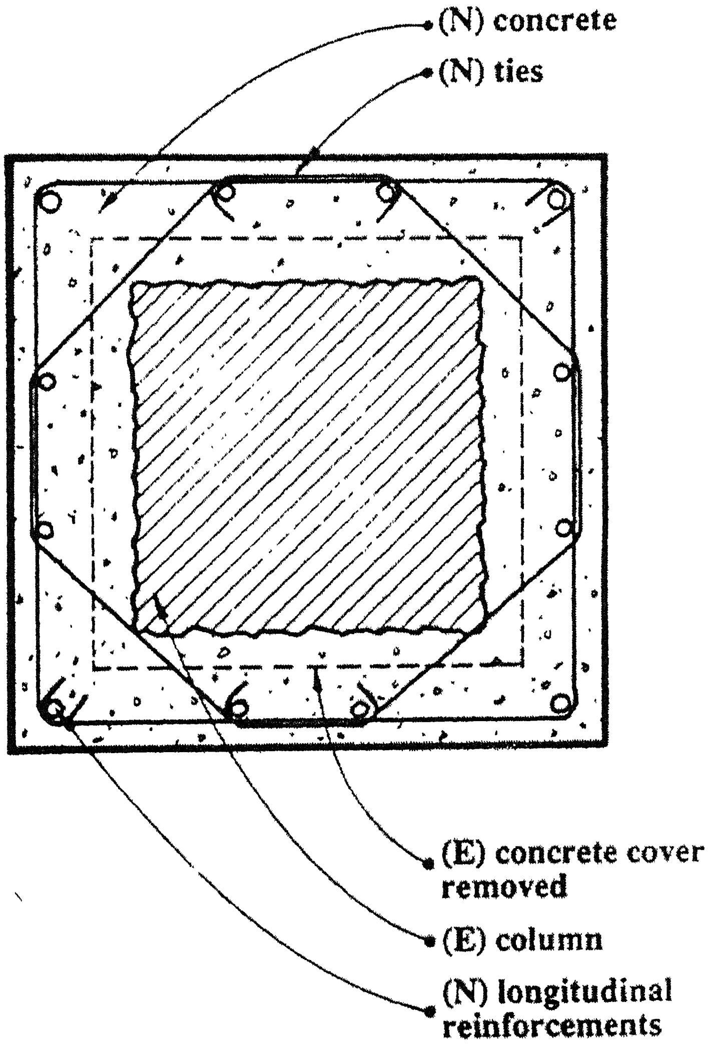


Fig. 4.1 : Column Jacketing [NEHRP, 1994]

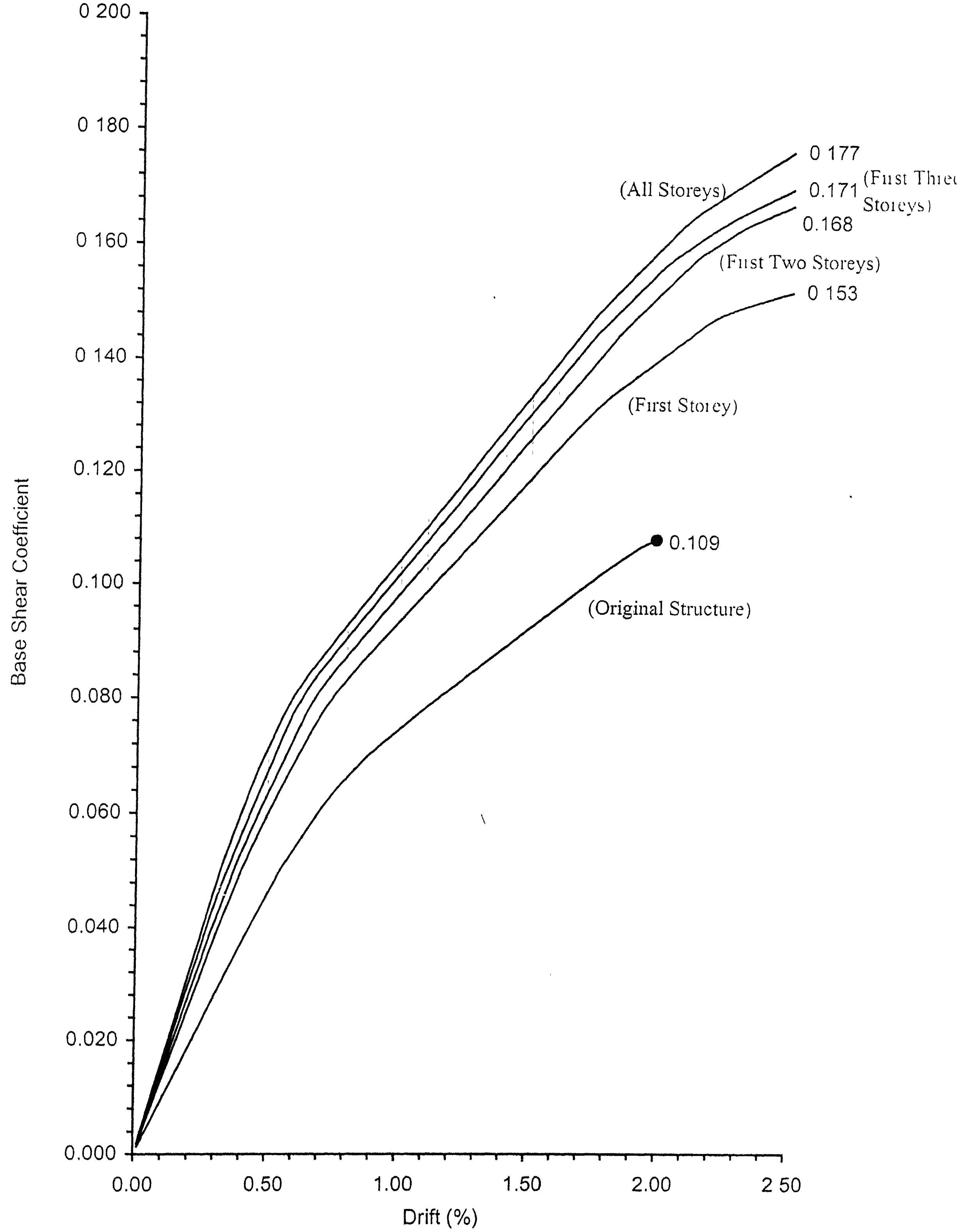


Fig. 4.2 : Force-Displacement Relations for Column Retrofitting

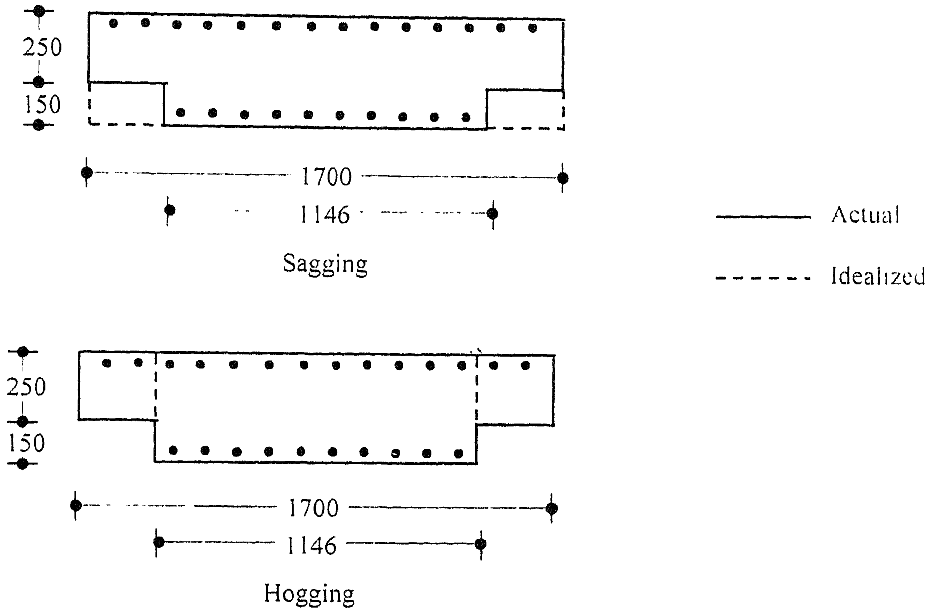


Fig. 4.3 : a) Idealization of Drop Panel Elements

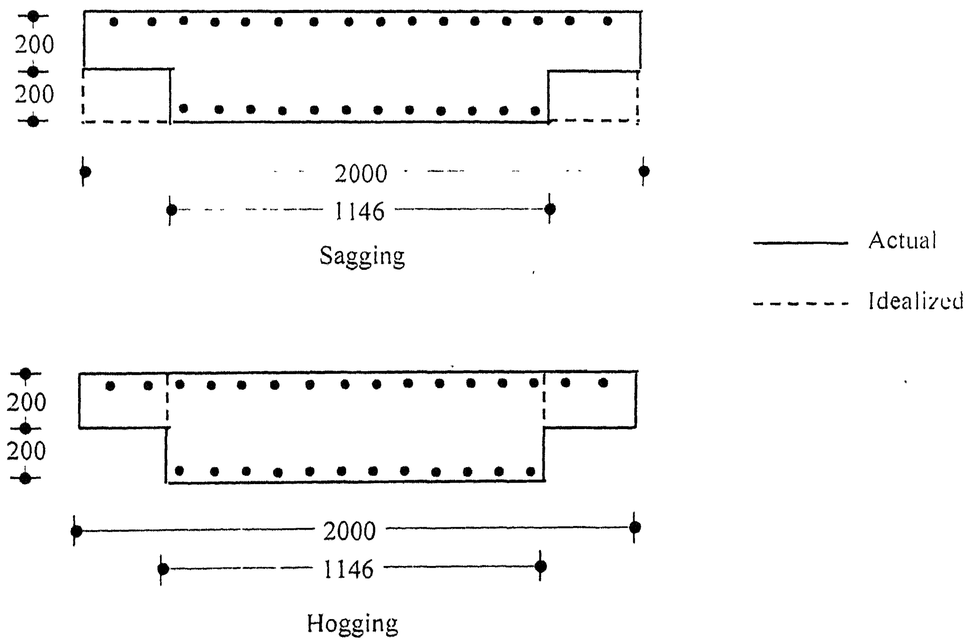


Fig. 4.3 : b) Idealization of Slab Elements

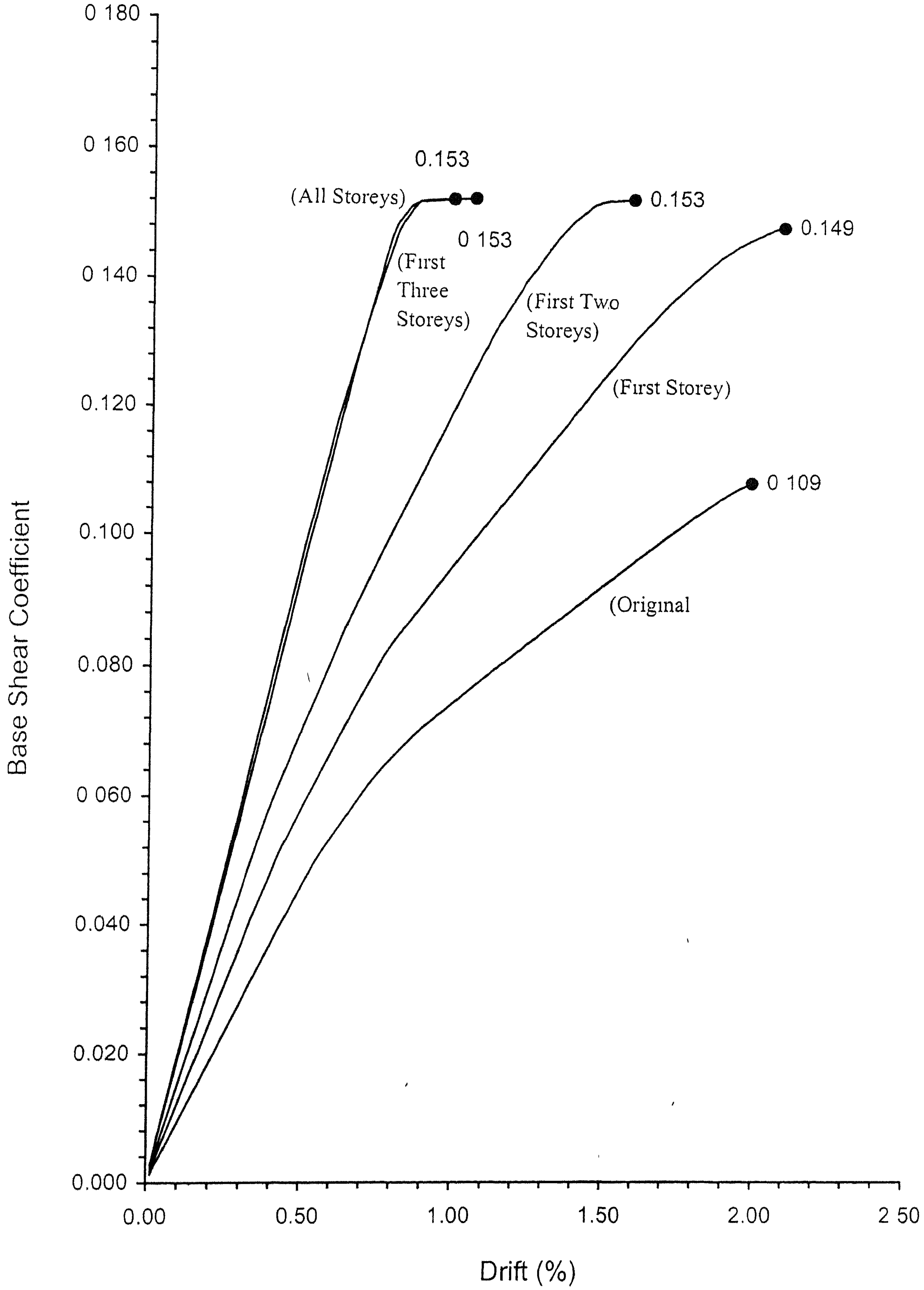


Fig. 4.4 : Force Displacement Relations for Beam Retrofitting

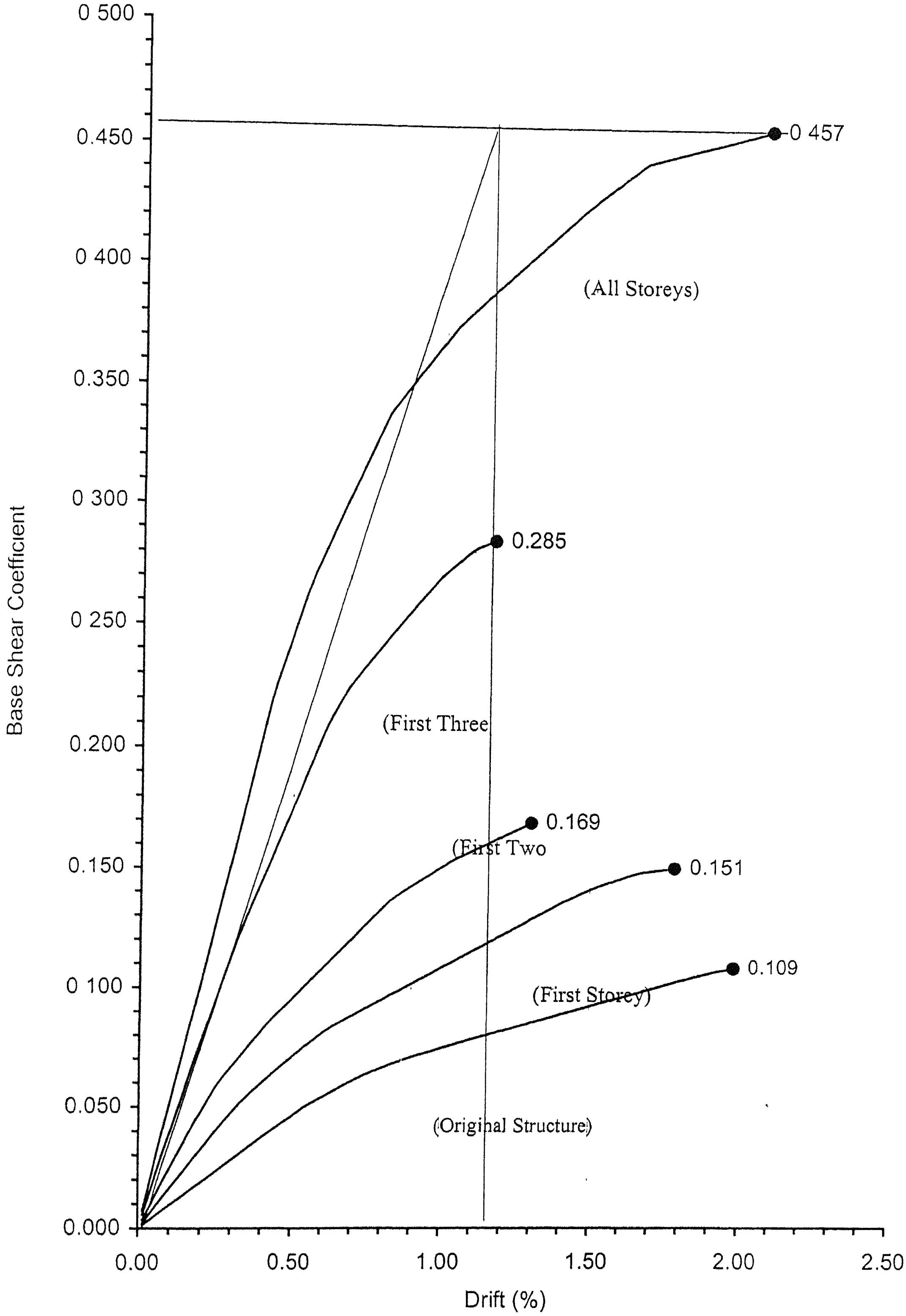


Fig. 4.5 : Force-Displacement Relations for Column and Beam Retrofitting

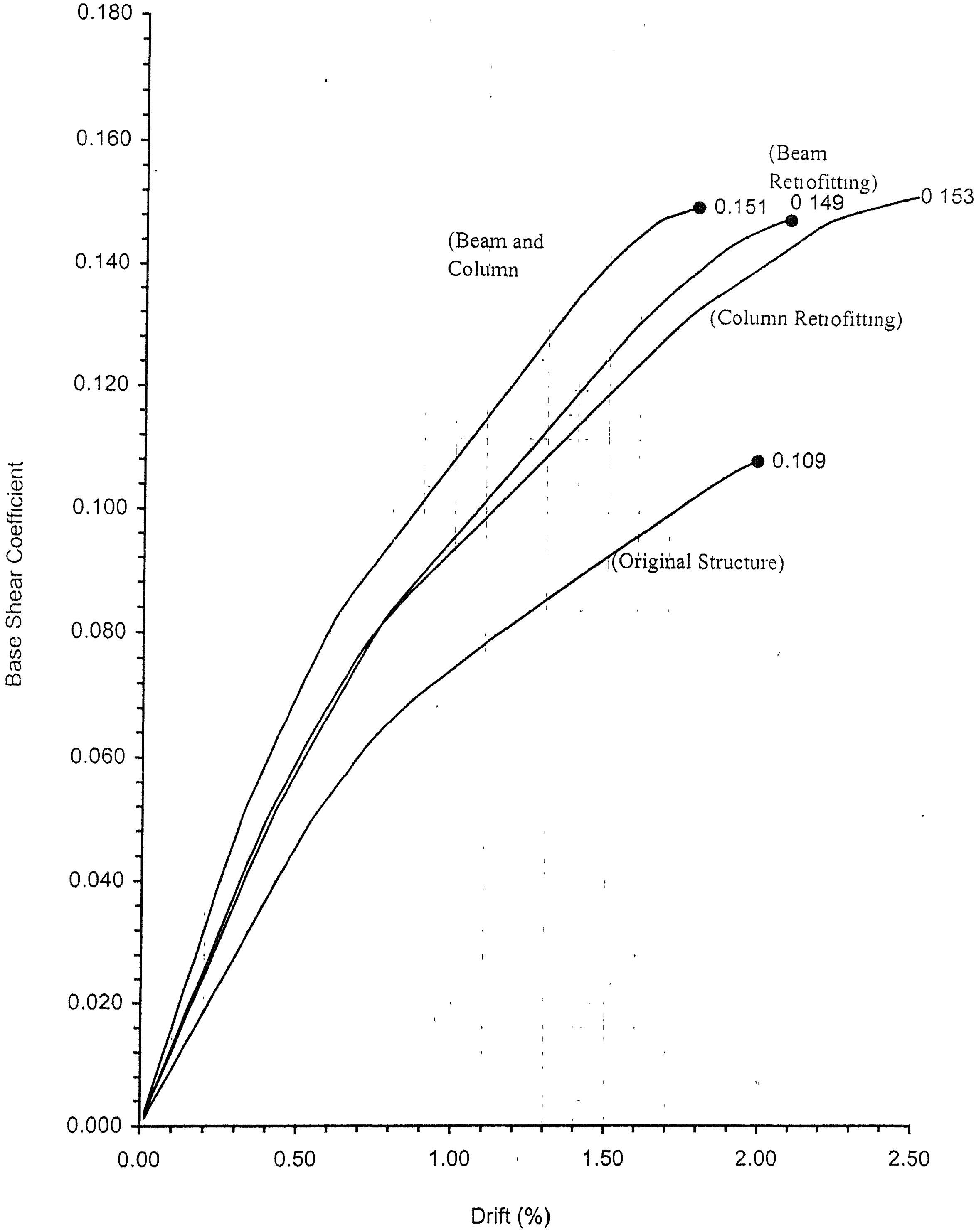


Fig. 4.6 : Force-Displacement Relations for Retrofitting of First Storey

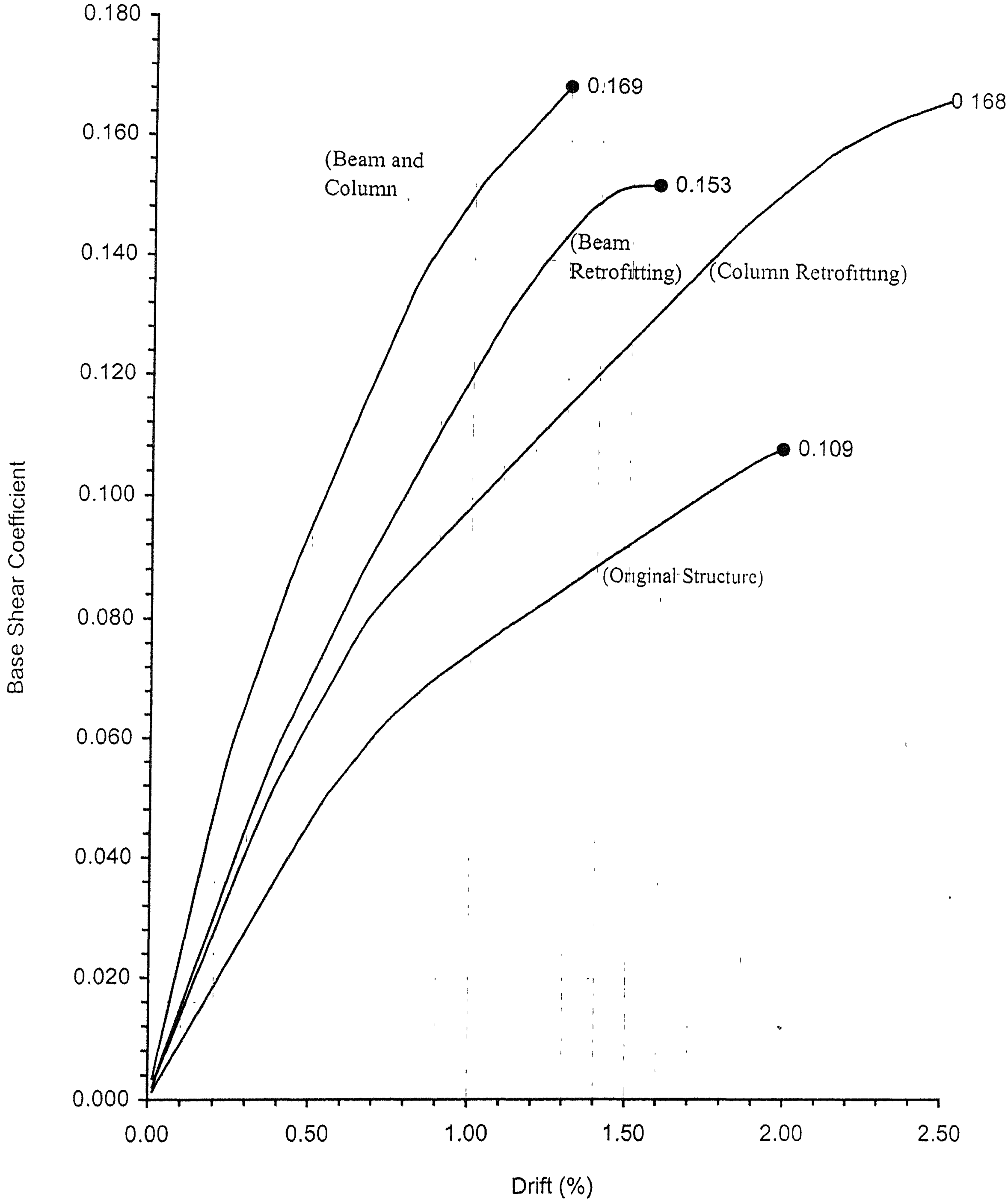


Fig. 4.7 : Force-Displacement Relations for Retrofitting of First Two Storeys

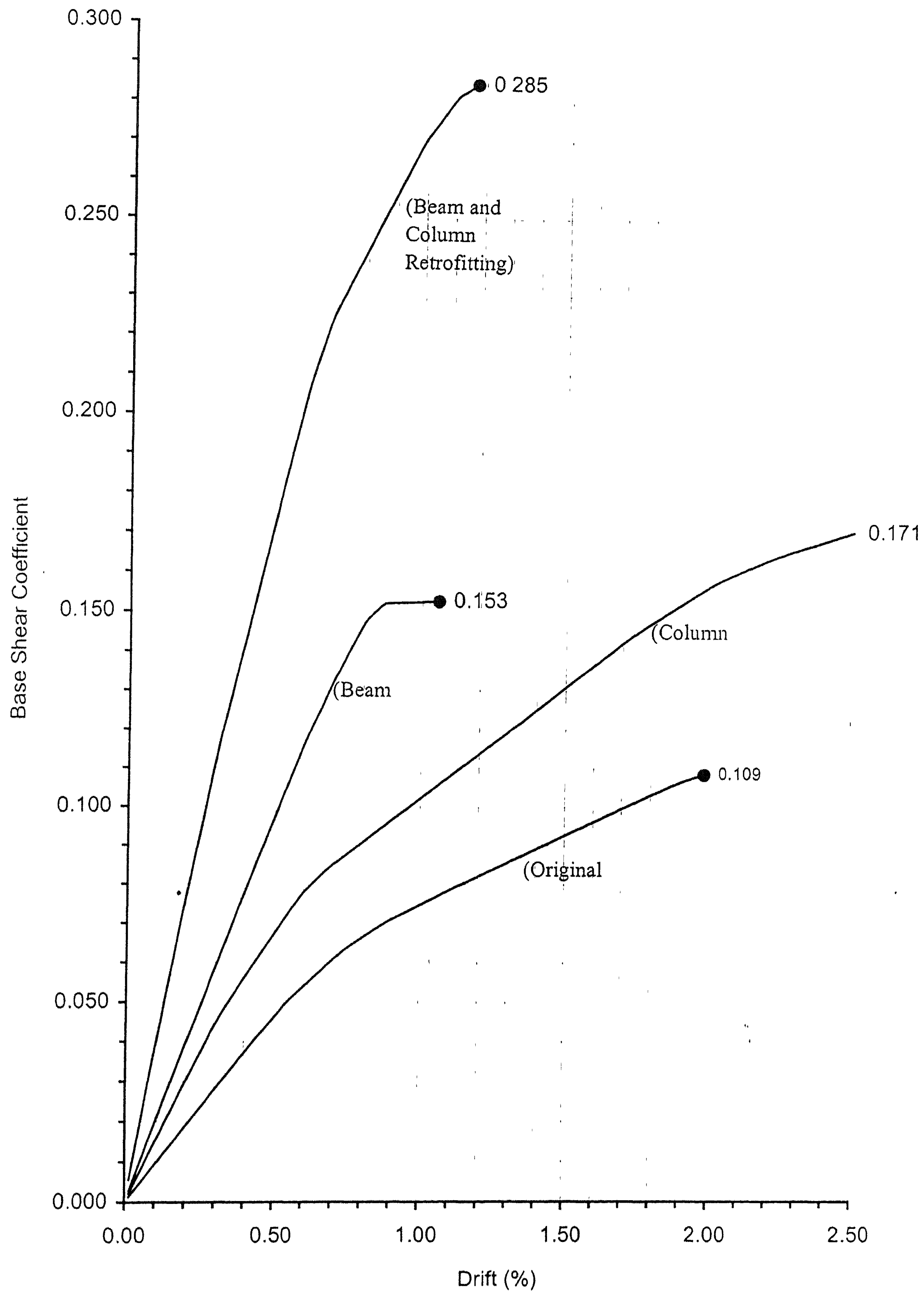


Fig. 4.8 : Force-Displacement Relations for Retrofitting of First Three Storeys

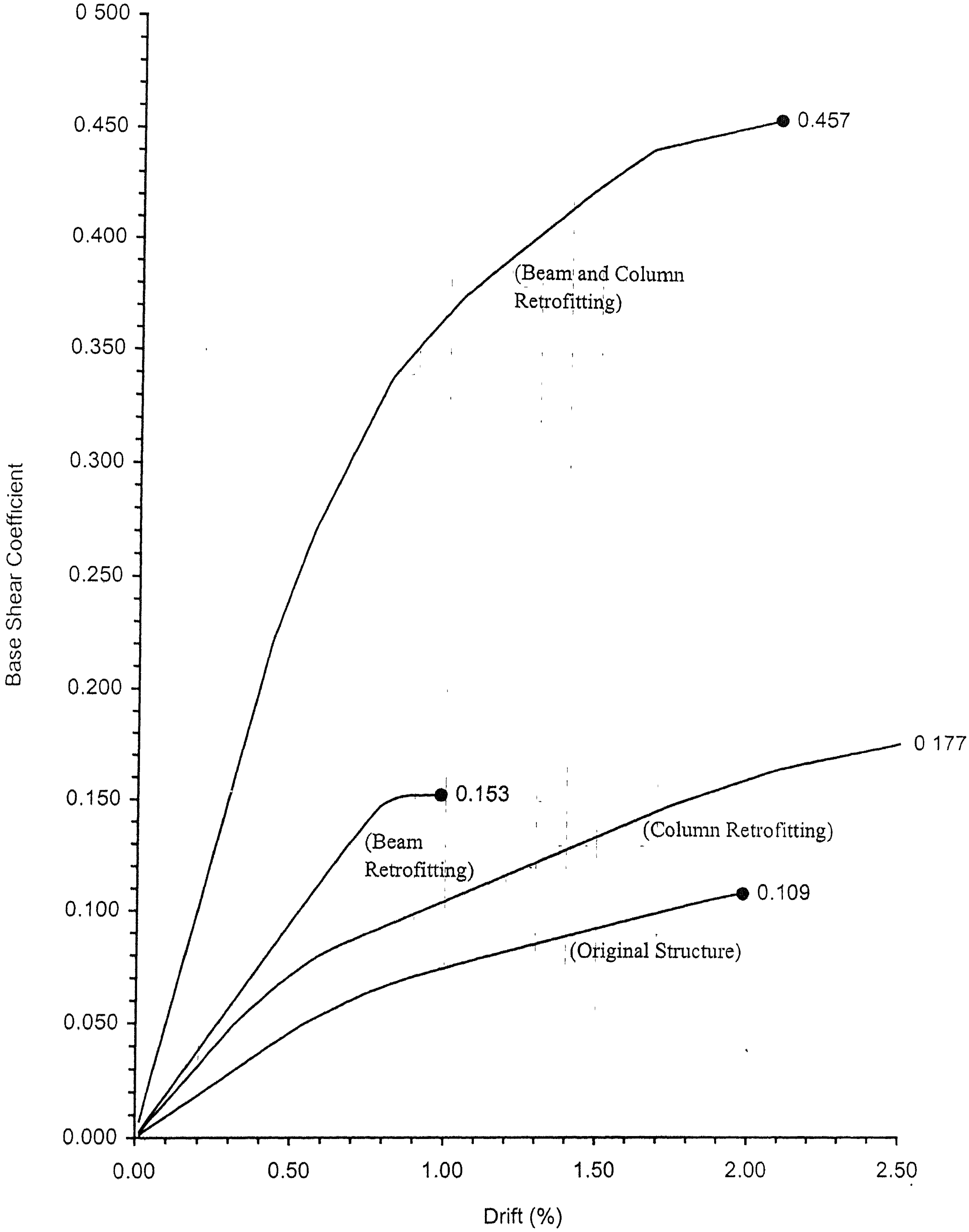


Fig. 4.9 : Force-Displacement Relations for Retrofitting of All Storeys

Chapter 5

Summary and Conclusions

Lateral response of an existing flat slab building is studied by nonlinear push-over analysis. A four-storey three-bay flat slab building, which is not designed for seismic loads, but designed for wind loads, was chosen for this purpose. The three-dimensional structure is replaced by two-dimensional frames consisting of original columns and slab of effective width [Dovich and Wight, 1994]. The push over analysis is carried out on the interior frame of this building. The design base shear coefficient due to the wind load on this building is found to be 0.028. The response of this building is compared with that of beam-column frames available in literature [Jain and Navin, 1995]. The change in behavior of this building due to the continuous column strip bottom reinforcement is also studied. From the above studies, it is observed that the seismic response of existing flat slab buildings is dangerously inadequate. Hence, many older flat slab buildings will urgently need retrofitting.

The response of flat slab buildings with retrofitting by different schemes a) column jacketing; b) addition of extra beam and c) column jacketing and addition of extra beam is studied. In each of these three schemes, the push over analysis is carried out for retrofitting of first storey, first two storeys, first three storeys and all storeys.

The important conclusions based on the above studies are as follows.

a) Performance of Existing Flat Slab Buildings

1. The existing non-seismically designed flat slab structures have much lower stiffness and drift capacity than in conventional beam-column frames, while the overstrength of quite comparable.

2. The yielding starts at very low drift of 0.5%, when the overstrength is only 1.6.
3. Significant softening takes place in the slab due to yielding in sagging, because of absence or insufficiency of bottom reinforcement.
4. The force-displacement relationship of these buildings hardly shows any ductility.

b) Performance of Existing Flat Slab Buildings with Continuous Column Strip Bottom Reinforcement

1. Initial stiffness is same as that of the original building.
2. The first yield is delayed from 0.5% drift in original building to 0.86% drift in this case. The overstrength at first yield increased significantly from 1.6 to 2.8.
3. The major concern is that the yielding in sagging reduced significantly.
4. The overstrength increased by about 10% from that in the original building.
5. The drift capacity reduced to 1.6% from 2% in original building.
6. No improvement in ductility is found.

c) Retrofitting with Column Jacketing

1. The overstrength increased by 40%, 54%, 57% and 62% and the initial stiffness increased by 41%, 52%, 63% and 72% with respect to the original building, as the number of retrofitted storeys increase from one to four.
2. The yielding in sagging is significant as in the original building.
3. The first yield started earlier around 0.31% while in the original building it started at 0.5%.
4. Column damages shifted to unretrofitted upper storeys from retrofitted bottom storeys.

5. No column failures are observed within the 2.5% drift in all cases of column jacketing. Hence, drift capacity is improved.

d) Retrofitting with Addition of Extra Beam

1. The overstrength increased by 37% in case of first storey retrofitting and 40% in all other cases, with respect to the original building.
2. The initial stiffness increased by 37% in case of first storey retrofitting, 68% in case of first two storeys of retrofitting, and 118% in case of first three and all storeys of retrofitting, with respect to the original building.
3. The drift capacity increased in case of first storey retrofitting by 5%, while it reduced by 20%, 47%, and 50% in case of first two storeys, first three storeys and all storeys retrofitting, respectively.
4. The yielding in sagging was limited to unretrofitted floors only.

e) Retrofitting with Column Jacketing and Addition of Extra Beam

1. The overstrength increased by 38%, 55%, 162% and 320% and the initial stiffness increased by 78%, 167%, 333% and 467% with respect to the original building, as the number of retrofitted storeys increase from one to four.
2. The drift capacity reduced by 10%, 34%, and 40% in case of first storey, first two storeys, and first three storeys, respectively, while in case of all storeys retrofitting drift capacity increased by 5% with respect to the original building.
3. The yielding in sagging was limited to unretrofitted floors only as in case of beam retrofitting.

f) Comparison of Retrofitting Schemes with Number of Storeys of Retrofitting

1 The first storey retrofitting gives similar overstrength for all the three schemes of retrofitting with more drift capacity in column jacketing. Hence column retrofitting of ground storey can be a very good cost effective technique.

2. The first two storeys retrofitting gives somewhat similar overstrength for all cases. The column retrofitting gives larger drift capacity, while the other two schemes have the advantage of lesser yielding in sagging.

3. In the first three storeys retrofitting, the beam plus column retrofitting gives very high strength and stiffness than the other two schemes, but drift capacity is less.

4. The all storeys retrofitting shows highest overstrength and stiffness among the all available combinations. The drift capacity in this case is similar to that of the original structure.

The program used in this study does not allow the M-θ curve of elements to have a failure point. However, the drift at which 'θ_u' is reached in column elements is determined by another program, to find the occurrence of column failure.

In future study, the experimental investigation of slab-column connection under lateral loading is recommended. It will be interesting to study analytically the effect of edge beam on overall behavior of structure. The improvement in behavior due to retrofitting by steel bracings is also of interest.

List of References

ACI 318-95, (1996) Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95), American Concrete Institute, Farmington Hills, MI, USA.

Alexander, S. D. B. and Simmonds, S. H., (1987) "Ultimate Strength of Slab-Column Connections," *ACI Structural Journal*, Vol. 84, No. 3, pp. 255-261.

ASCE-ACI Committee 426, (1974) "The Shear Strength of Reinforced Concrete Members- Slabs," *Journal of the Structural Division*, ASCE, Vol. 100, No. ST8, pp. 1543-1591.

Cano, M. T., and Klingner, R. E., (1988) "Comparison and Analysis Procedures for Two-Way Slabs," *ACI structural Journal*, Vol. 85, No.6, pp. 597-608.

Corley, W. G., Sozen, M. A., and Siess, C. P., (1961) "The Equivalent Frame Method Analysis for Reinforced Concrete Slabs," *Structural research Series No 218*, Civil Engineering Department, University of Illinios, Urbana, 168 pp.

Corley, W. G., and James O. J, (1971) "Equivalent Frame Analysis for Slab Design," *ACI Journal*, November 1970, pp. 875-884. Discussion by Eberhardt and Hoffman, Ti Huang, Jofriet and Handa, and Closure, pp. 397-401.

Cruzado-Martinez J, A., Qaisrani, A. N., and Moehle, J. P., (1994) "Post-Tensioned Flat-Plate Slab-Column Connections Subjected to Earthquake Loading," *5th US National Conference on Earthquake Engineering*, Chicago, Vol.2, pp.139-148.

Darvall, P., and Allen, F., (1984) "Lateral Load Effective Width of Flat Plates with Drop Panels," *Journal of American Concrete Institute*, Proceedings Vol.81, No. 6, pp.613-617.

Dovich, L. M., and Wight, J. K., (1994) "Lateral Response of Nonseismically Detailed Reinforced Concrete Flat Slab Structures," *Report no. UMCEE 94-30*, Department of Civil and Environmental Engineering, The University of Michigan. USA.

Durrani, A. J., and Diaz, A. J., (1992) "Seismic Resistance of Fiber-Reinforced Slab-Column Connections," *Tenth World Conference on Earthquake Engineering*, Spain, Vol.6, pp.3113-3116.

Durrani, A. J., Du, Y., and Luo, Y. H., (1995) "Seismic Resistance of Nonductile Slab-Column Connections in Existing Flat-Slab Buildings," *ACI Structural Journal*, Vol. 92, No. 4, pp. 479-487.

Elias, Z. M., (1983) "Lateral Stiffness of Flat Plate Structures," *Journal of the American Concrete Institute,* Proceedings Vol. 80, No. 1, pp. 50-54.

Elias, Z. M., Georgiadis, C., (1979) "Flat Slab Analysis Using Equivalent Beams," *Journal of American Concrete Institute,* Proceedings Vol. 76, No. 10, pp. 1063-1078.

Goel, S. C., and Masri, A. C., (1994) "Seismic Strengthening of RC Slab-Column Frames with Steel Bracing," *5th US National Conference on Earthquake [illegible]*, Chicago, Vol.3, pp.789-798.

Hanson, N. W., and Hanson, J, M., (1968) "Shear and Moment Transfer Between Concrete Slabs and Columns," *Journal of the Portland Cement Association Research and Development Laboratories*, Vol. 10, No. 1, 16 pp.

Hayes, J. R., Fouch, D. A., and Brady, P. A., (1996) "Experimental Investigation of the use of Visco-Elastic Dampers to Rehabilitate a RC Flat Slab Structure," Paper No.420, *Eleventh World Conference on Earthquake Engineering*, Mexico.

Hawkins N. M., (1973) "Shear Strength of Slabs with Moments Transferred to Columns," Shear in Reinforced Concrete, *ACI publication SP-42,* American Concrete Institute, Detroit, pp. 817-846.

IS:456 (1978) Indian Standard Code of Practice for Plain and Reinforced Concrete, Bureau of Indian Standards, New Delhi, India.

IS:1893 (1984) Indian Standard Code of Practice for Earthquake Resistant Design of Structures, Bureau of Indian Standards, New Delhi, India.

IS:13920 (1993) Indian Standard Code of Practice for Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces, Bureau of Indian Standards, New Delhi, India.

Jain, S.K., and Navin, R., (1995), "Seismic Overstrength in Reinforced Concrete Frames" *Journal of Structural Engineering,* ASCE, Vol.121, No.ST3, pp 580-585.

Khannan, A. E., and Powell, G. H., (1972) "Dynamic Response Analysis of Inelastic Structures (DRAIN-2D)," University of California, Berkeley, USA.

Mandal, P., and Jain, S. K., (1993) "Curvature Ductility of RC Sections With and Without Seismic Confinement," Department of Civil Engineering, Indian Institute of Technology, Kanpur, India.

Mitchell, D., and Cook W. D., (1984) "Preventing Progressive Collapse of Slab Structures," *Journal of Structural Engineering*, Vol. 110, No.7, pp.1513-1532.

Mitchell, D., and Michael, C. P., (1974) "Diagonal Compression Field Theory - A Rational Method for Structural Concrete in Pure Torsion," *ACI journal*, Proceedings Vol. 71, No. 8, pp. 396-408.

Moehle, J. P., (1988) "Strength of Slab-Column Edge Connections," *ACI Structural Journal*, Vol. 85, No. 1, pp. 89-98.

Moehle, J.P., Diebold, J.W., and Zee, H. L., (1984) "Experimental study of a flat-plate building model," *Eighth World Conference on Earthquake Engineering*, San Francisco, Vol.6, pp.355-362.

NEHRP (1992), *NEHRP Hand Book for Seismic Rehabilitation of Existing Buildings*, Federal Emergency Management Agency, FEMA-172, p.24.

Pan, Austin, D., and Moehle, J. P., (1992) "An Experimental study of Slab-Column Connections," *ACI Structural Journal*, Vol. 89, No. 6, pp. 626-628.

Park, R., and Islam, S., (1976) "Strength of Slab-Column Connections with Shear and Unbalanced Flexure," *Journal of Structural Division*, vol. 102, No. ST9,pp. 1879-1901.

Park, R., and Paulay, T., (1975) *Reinforced Concrete Structures*, John Wiley and Sons, New York, 248p.

Pecknold, D. A., (1975) "Slab Effective Width for Equivalent Frame Analysis," *Journal of the American Concrete Institute*, Proceedings Vol. 72, No. 4, pp. 135-137.

Rai, D. C., Goel, S. C., and Firmansjah, J., "A General Purpose Computer Program for Static and Dynamic Analysis of Two-Dimensional Structures (SNAP-2DX)," Ann Arbor, University of Michigan, USA.

Robertson, I. N., and Durrani, A, J., (1992) "Gravity Load Effect on Seismic Behavior of Interior Slab-Column Connections," *ACI Structural Journal*, V. 89, No. 1, pp. 37-45.

Robertson, I. N., Durrani, A.J., and Rodriguez, M. E., (1988) "Seismic Behavior of Multiple-Connection RC Flat Slab Assemblies," *Ninth World Conference on Earthquake Engineering*, Tokyo, Vol.4, pp.571-576.

Salakawy, E. F., Polak, M. A., and Soliman, M. H., (1998) "Investigating the behaviour of RC flat slabs with openings," T201-4, *Structural Engineering World Conference*, San Francisco.

Shen Ju-min, Li De-sen, and Fu Ping-zun, (1984) "Experimental Study on Earthquake Response of Models of Precast RC Flat Slab Building Structures," *Eighth World Conference on Earthquake Engineering*, San Francisco, Vol.6, pp.741-748.

Tagoes, J. A., and Tsonos, A. G., (1996) "Punching Strength Decay of Slab-Column Connections Under Seismic Loading," Paper No.654, *Eleventh World Conference on Earthquake Engineering*, Mexico.

Tang, X., and Goel, S. C., (1988) "DRAIN-2DM Technical Notes and User's guide," Ann Arbor, University of Michigan, USA.

Vanderbilt, M. D., (1972) "Shear Strength of Continuous Plates," *Journal of Structural Division*, Vol. 98, No. ST5.

Vanderbilt, M. D., and Corley, W. G., (1983) "Frame Analysis of Concrete Buildings," *Concrete International - Design and Construction*, V. 5, No. 12, pp. 597-608.

Zee, H. L., and Moehle, J. P., (1984) "Behavior of Interior and Exterior Flat Plate Connections Subjected to Inelastic Load Reversals," *Report No. UCB/EERC-84/07, Earthquake Engineering reasearch Center*, University of California, Berkeley, California.